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Reconstruction of the Ice Skating Rink at the University of Illinois*

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SYNOPSIS

The paper describes the reconstruction of a large ice skating rink floor, made necessary by the failure of the brine pipes in the original floor.

The original floor system consisted of a base concrete sub-floor covered with a 2 in. sand cushion overlaid with a 3¾ in. concrete finished floor containing the refrigerating brine pipe coils. The finished concrete floor was cast in 6 ft. wide sections with brass contraction joints between the sections. Steel turnings mixed with the concrete to increase the conductivity believed to have produced porous concrete.

Failures first occurred where wood wedges had been left in the concrete between the brass contraction joints and adjacent pipes. The corrosion and pitting of the pipes adjacent to the wedges was thought to have been due to galvanic action, the moist wood forming the return path for the electrolytic current as it passed from the pipe to the brass. The calcium chloride brine released from these early leaks spread rapidly throughout the sand cushion causing rapid deterioration of the entire floor system. Comparison of construction details and failure of other major ice skating rinks revealed possibility of failure from other sources than the galvanic action created by the local battery.

The new floor system was designed to eliminate, insofar as possible, all apparent causes of past rink failures. The sand cushion was eliminated and the thickness of the concrete was increased to 5⅝ in., monolithic, placed and finished in a continuous operation without expansion, contraction, or construction joints, eliminating the brass strips or other dissimilar metals. Uncoated steel pipe was used free from mill scale, oil, or foreign matter that would destroy bond with the concrete. The pipes were joined by oxy-acetylene welds, and supported on rolled steel tee sections cast in the concrete. The concrete was designed for maximum density and minimum shrinkage and carefully placed.

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1. GENERAL INFORMATION

The mechanically refrigerated indoor ice skating rink of the University of Illinois was built in the spring of 1931 and was opened for public use in November of that year. The first failure appeared as leaks in the brine-circulating coils contained in the concrete floor during 1934. Failure progressed so rapidly during 1935 and 1936 that, by 1937, with more than one-third of the coils out of service, it was decided that further repairs were not justified and that it would be more economical to renew the entire floor.

This work was started Nov. 1, 1937, and the rink was opened for public use on Feb. 1, 1938. The total cost of the project was $26,444.47 or $1.097 per sq. ft. of floor, including royalty on the patented floor construction and certain changes in main headers to improve circulation of brine and eliminate some former operating difficulties.

2. CONSTRUCTION OF ORIGINAL FLOOR SYSTEM

The construction of the original floor system, known as the expansion floating type, had been used in most of the major rinks throughout the country. The skating arena was rectangular in plan with rounded corners, (radius 20 ft.6 in.) 126 by 194 ft. The total area was 24,099 sq. ft.

A header tunnel was constructed along the side nearest the engine room. A three-header system was used to equalize friction in the hairpin coils extending across the rink. A pipe trench was located on the opposite side and both ends of the rink to house other utilities.

The original construction (Fig. 1) consisted of a concrete subfloor 6 in. thick, over an 18-in. cinder fill, containing a system of tile subdrainage.

Creosoted wood sleepers 1¾ x 2¾ in. were laid lengthwise of the rink on 6 ft. centers over the subfloor but were not fastened to it. On these sleepers were 1-in. round steel bars for pipe supports. Over the steel bars and crosswise of the rink 1¼ inch standard black steel pipe was placed 4½ in. on center, held in place with straps nailed to the wood sleepers.

The pipes were in 40 ft. lengths and all intermediate joints were oxyacetylene welded. They were connected alternately on one side of the rink to the supply and return header and on the opposite side with hairpin return bends. Connections to the headers were made with screwed fittings with a valve on each pipe. A continuous air vent was provided and connected to the supply side of each coil at the header. Individual air-vent cocks were installed on each return coil in the header tunnel.

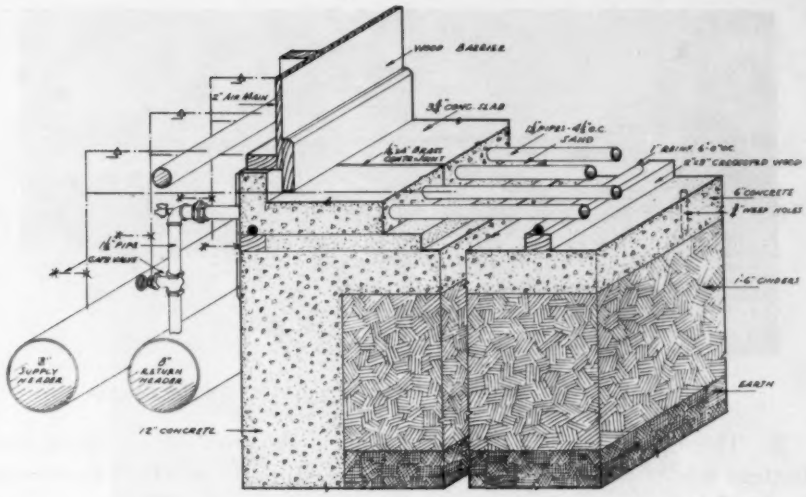

FIG. 1—SECTION THROUGH OLD FLOOR

After the pipes were placed, tested and leveled, clean coarse sand was filled and tamped in, level with the top of the wood sleepers. The concrete floor was then placed directly upon the sand base, covering the pipes 1¼ in. The overall thickness of the finished floor was approximately 3¾ in.

The mixture for the concrete floor consisted of 1 part cement, 1½ parts sand and approximately 2 parts metal turnings. The floor was placed in alternate sections 6 ft. wide crosswise of the rink with 1/16 in. x 4 in. brass contraction joint strips between sections. Intermediate sections were placed later. The surface of each section was troweled to a smooth finish and the high spots ground off with a carborundum machine.

A curb 4 in. wide and 3 in. high was built around the edge of the rink, integral with the floor.

3. FAILURE OF ORIGINAL FLOOR SYSTEM

The actual cause of the first failure in the system is not known. After the first leak developed, the failure was progressive and rapid.

Several sections of the old floor were removed (Fig. 2) in as large units as possible, and the concrete was broken away carefully for examination of the brine pipes, pipe supports, quality of concrete, contraction joint strips, sand bed, etc. This disclosed several interesting facts:

1. The failure in the brine pipes was due to external corrosion. Little, if any, corrosion was detected on the inside of the pipes.

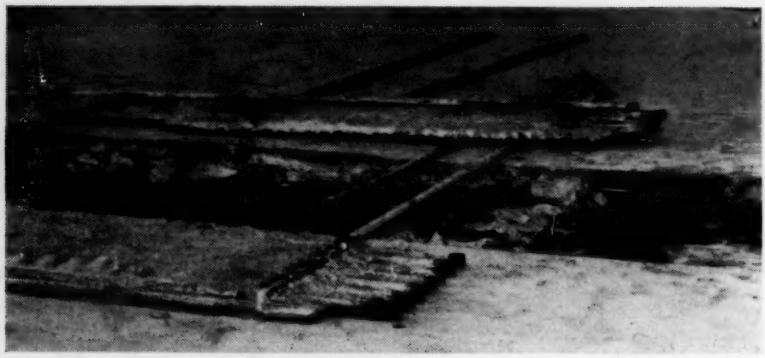

FIG. 2—SECTIONS OF OLD FLOOR WERE REMOVED

2. The extent of the corrosion was more pronounced along the bottom side of the pipe and over the pipe supports or where there was no bond between concrete and steel.

3. Failure was more pronounced in the vicinity of the first leak.

4. The surface of the concrete was dense and hard. The main body of concrete was porous and marked by frequent spirals of steel turnings increasing the porosity. Over the supports the pipes were practically exposed on the bottom side (Fig. 3). Shrinkage of concrete away from the pipes, with resulting loss of bond, was in evidence adjacent to the pipe supports. Corrosion was more pronounced in these porous areas. Where the pipes were completely encased and where there was good bond between concrete and steel, little, if any, corrosion had taken place.

5. The sand cushion was found to be saturated with calcium chloride, more concentrated in the vicinity of the early leaks, decreasing further away. At one end in the vicinity of the first leak, the sand contained 864 mg. calcium chloride per 100 gm. sand, in the center of the floor 106 mg , and at the opposite end 39 mg., indicating that the brine had spread throughout the sand.

6. There were definite electrical potentials between the brass strips and the steel pipes, indicating presence of a local battery created by the use of dissimilar metals with, first, moisture from the ice, and later, brine as the electrolyte. Voltmeter readings indicated potential of (1) 70 mv. from the pipe to ground with the pipe positive; (2) 20 mv. from brass strip to ground with the brass positive; and (3) 50 mv. between brass strip and pipe with the pipe positive. This was an ideal condition for electrolysis, although many authorities on corrosion stress that mere potential does not indicate that the galvanic action is alone responsible for the corrosion.

FIG. 3—CORRODED PIPES WERE EXPOSED ON UNDERSIDE

7. At irregular intervals 1 x 2 in. wedge-shaped wood blocks had been cast in the concrete between the brass joint strip and the adjacent pipe (Fig. 4). The wood blocks evidently had been used to brace and support the brass strip during construction and had not been removed. In each instance where the wood blocks were found, the side of the pipe in contact with the wood, was badly pitted, and in many places, the pipes were leaking at this point. Wood blocks had been found at the initial point of failure. The blocks, naturally good accumulators of conductive solution, either water or brine, could have produced a short circuit between the brass strips and the steel pipes.

8. The possibility of electrolysis from stray electric currents was checked and these were not found to be present.

Several factors contribute to the failure of metal by corrosion. The problem was to determine which of the foregoing faults were the controlling factors in order to eliminate them as far as possible before construction of the new rink.

A study was made of construction and failure of ten of the major rinks in the United States. All of the rinks constructed similarly to the University rink had experienced pipe failures in from three to

FIG. 4—PIPE BADLY PITTED IN CONTACT WITH WOOD BLOCKS USED TO SUPPORT BRASS STRIPS

six years after construction. Some had been completely replaced earlier, some were being replaced, and others were being repaired extensively, with complete replacement contemplated. The general design of all were similar, the difference in detail being in minor change in pipe supports, contraction joint strip, pipe material, and main header arrangement. Some had used 1 x 1 in. angles over the wood sleepers for pipe supports. Some had used extra heavy black steel pipe, some genuine wrought iron, some were welded together, and some were made up with hydraulic-type screwed couplings. Some had used precast asphalt contraction joints, some narrow strips of brass, and others strips of asphalt-saturated felt.

There was no apparent difference in life between the black steel pipe and genuine wrought iron or between the welded and screwed method of construction.

One rink had used black steel pipe with genuine wrought-iron couplings and corrosion had attacked both equally (Fig. 5).

In all the failures it was apparent that after the first leak developed, the failure was general, rapid and progressive.

Some attributed the first leak to screwed couplings, others to pin holes in welded joints, and one to fracture by falling object severing several of the pipes.

Some suggested the possibility of electrolysis from stray electric currents from adjacent electric street railways, although this factor had not been checked.

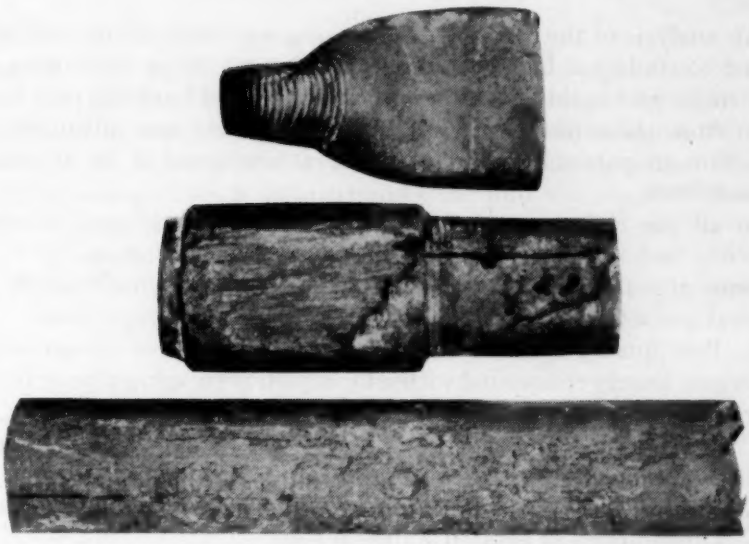

FIG. 5—CORROSION ATTACKED BOTH STEEL AND WROUGHT IRON
(top) black steel pipe; (centre) black steel pipe; genuine wrought iron coupling; (bottom) genuine wrought iron pipe

Some suggested the possibility of corrosion from acidic constituents that may have been leached from the sand, or from the concrete, or even from the cinders underlying the concrete subfloor.

It was noticed that in most rinks, failure appeared first at the points where the pipes entered the concrete, or where there was no bond between concrete and steel, or where pipes were imbedded in cork or were in contact with wood. From this, it was evident that the presence of oxygenated water or of moist air was the cause of part of the corrosion.

The original corrosion at the University rink was localized at the points where moisture could seep through the wood blocks between the brass strips and the pipes. The corrosion and pitting were thought to have been due to galvanic action, the moist wood forming the return path for the electrolytic current as it passed through the solution from the pipe to the brass. Since some rink failures had occurred where neither brass nor wood was present, it was concluded that corrosion was not due entirely to the presence of brass, and could have been caused by the presence of water alone in contact with the pipes.

In all rinks examined, it was thought that the presence of the coils of steel turnings in the aggregates had been contributing factors in the resultant porous concrete. There was also the possibility that the turnings contained some brass or dissimilar metal that would produce galvanic action.

An analysis of the products of corrosion was made at one rink and found to contain a high percentage of sulphur. Since there were no cinders present in this rink, it was thought the acid leachings may have been from the cement or aggregate. This theory was advanced by one firm in particular; however, others considered it as of minor consequence.

In all the rinks examined, where there was good bond between concrete and the pipes, there was no evidence of corrosion.

From this investigation, the possible causes of the rink failure in general are set forth as follows in the order of their importance:

1. Poor quality concrete containing steel turnings or other foreign aggregate loosely compacted without completely encasing pipe or being thoroughly bonded thereto.
2. Leaks in pipes, accidental or otherwise, from screwed joints, faulty welds or pin holes.
3. Electrolytic or galvanic action from use of dissimilar metals.
4. Direct action of aerated water on steel.
5. Acceleration of item 4 where pipes are not completely encased or where the concrete is porous, full of voids and air pockets, and no bond between concrete and steel.
6. Acceleration of item (4) in the presence of brine, which may occur from accidental leaks or mechanical faults.
7. Action of any acidic constituents that may be leached from the sand, or from the concrete, or from cinders underlying the concrete.
8. Electrolysis from stray electric currents.

4. TEST OF WELDED JOINTS

In order that some definite conclusion could be reached as to the relative merits of the fusion weld (resistance butt weld) vs. the oxyacetylene and arc weld for pipe joints, the following procedure was observed.

Test samples were prepared by both the University and an outside firm specializing in refrigerator work. The samples consisted of 1-in. wrought-iron pipe and 1-in. standard black steel pipe sections. Each section consisted of two pieces of the same material approximately 6 in. long and joined by welding.

Each and all specimens were tested for tensile strength in the same manner. Steel plugs of 1-in. diameter and 3-in. long were inserted in each end of the various pipe sections. This was to provide a means for gripping specimens in the tensile testing machine.

All specimens were tested in tension until failure occurred. The results including identifying features of each specimen are tabulated (Table 1).

Results and Discussion:

All the fusion weld specimens failed at the weld and left a clear-cut exposure of the original adjoining faces. There was only slight indication of necking-down of the pipe, which would indicate the weld strength to be considerably below the pipe wall strength. This is further indicated by a comparison of the unit stresses developed by the same materials welded by other processes.

The failure of arc and oxyacetylene specimens in or at the weld indicated that it was due to workmanship rather than to the method.

TABLE 1—TEST OF WELDED JOINTS

Specimen No.	Prepared by	Type of Weld	Pipe	C. S. Area of Wall sq. in.	I. D. In.	O. D. In.	Brkg. Load Lbs.	Unit Stress Lbs. per sq. in.	Failure
1	Outside Firm	Arc	1" W.I.	.503	1.05	1.32	22980	45600	at edge of weld
2	Outside Firm	Acy.	1" W.I.	.503	1.05	1.32	24780	49200	in pipe wall
3	Outside Firm	Fus.	1" W.I.	.503	1.05	1.32	21790	43250	in butt weld
4-1	U. of I.	Acy.	1" Steel	.508	1.04	1.315	28200	55400	in pipe wall
4-2	U. of I.	Acy.	1" Steel	.519	1.04	1.32	28480	54900	in pipe wall
5-1	U. of I.	Arc.	1" Steel	.495	1.03	1.30	22960	46400	in weld
5-2	U. of I.	Arc	1" Steel	.505	1.03	1.305	28460	56500	in pipe wall
6-1	U. of I.	Arc	1" Steel	.47	1.07	1.32	27670	58800	in weld
6-2	U. of I.	Arc	1" Steel	.5025	1.05	1.32	26860	53500	in weld
7	Outside Firm	Acy.	1" Steel	.511	1.02	1.30	28840	56500	in pipe wall
8	Outside Firm	Fus.	1" Steel	.495	1.03	1.30	25500	51600	in butt weld
9	Outside Firm	Arc.	1" Steel	.511	1.02	1.30	29220	57200	in pipe wall
Extra	*Specimens*								
10	Outside Firm	Fus.	1¼" W. I.	.698	1.33	1.63	27740	39700	in butt weld
11	Outside Firm	Acy.	1¼" W. I.	.661	1.36	1.64	34990	53000	at edge of weld
12	Outside Firm	Arc	1¼" W. I.	.661	1.36	1.64	30060	45500	at edge of weld

Symbols
Arc — Arc Weld.
Fus. — Fusion Weld.
Acy. — Oxyacetylene Weld.
W.I. — Wrought Iron

November 16, 1937.
Machine No. 6
Operator—T. & A. M. Department
Recorder: W. W. H.

This was borne out by some arc and gas-welds exceeding the strength of the pipe walls. Furthermore, arc welds appear the more susceptible to burning, and hence, inconsistency in strength and bonding.

All the oxyacetylene welds developed high unit stresses in specimen failure. Only one oxyacetylene weld failed near the weld. In this specimen the pipe wall was near failure as indicated by the necking-down on both sides of the weld, and burning of the metal was apparent.

Conclusion:

The conclusions reached are:

1. The fusion weld is unsatisfactory because of its mechanical weakness and hence its susceptibility to seepage and corrosion.

2. The arc and oxyacetylene welds, when made properly, are superior to the fusion weld.

3. The oxyacetylene weld is preferable to the arc weld because with average equipment better control of the welding temperature can be obtained with consequent strength, bonding, and consistency of results.

5. SUGGESTED METHODS FOR PREVENTING CORROSION

To prevent corrosion these measures were suggested:

1. Select a concrete mixture with aggregates carefully selected and sized to produce a dense impervious mass that can be readily puddled in and completely around pipes and fixtures without producing voids and that will harden with a minimum of shrinkage.

2. Design a pipe support that has a minimum of surface contact with the pipes and around which the concrete may readily be puddled. The fewer the contacts of the pipe with other steel members, the better will be the results. Although it has never been used, a pipe support that could be removed after the concrete has been placed would be advisable, permitting the pipes to settle into the mass by their own weight as the concrete sets, thus preventing shrinkage away from the pipes and loss of bond.

3. The sand cushion is of little value and should be omitted. It provides a porous channel through which, in case of accidental leak, brine may spread readily to other parts of the floor.

4. Pipe should be carefully selected, inspected and tested for faults in manufacturing. It may be of black steel or genuine wrought iron, preferably of lap-weld construction in long lengths without mid-welds. Past failures indicate that little, if any, advantage is to be gained by the use of genuine wrought-iron, as both genuine wrought-iron and steel pipe failed under similar conditions. The additional cost of wrought-iron does not seem warranted. If extra precaution is thought necessary, extra heavy black steel pipe would be preferable because of additional wall thickness. The pipes should not be coated inside or outside and should be free from oil, mill scale or rust, that will prevent good bond with concrete. They should be thoroughly cleaned of all foreign matter before the concrete is placed.

5. Field joints should be as few as possible and should be welded. The welds should be uniform and free from pin holes, pockets, etc., and all loose scale removed.

6. A monolithic concrete slab with integral top is preferable to one laid in sections. The slab should be reinforced at right angles to the line of pipes to prevent cracking from expansion and contraction.

7. Brass or other dissimilar metals should be avoided entirely. Mixing steel turnings with concrete apparently has no advantage and may cause trouble. The heat conductivity of such concrete is not increased and there is always possibility of seepage of water and the presence of air pockets. Furthermore, there is a good possibility that there is a small percentage of brass in the turnings.

8. The use of protective coatings of asphaltum or bituminous materials on the pipe, or anything that will lessen the bond between concrete and steel, should be avoided. It is generally accepted that some types of coatings are excellent when there is assurance that they are uniform and that cracks will not develop. Whenever there is a failure of the coating, however, there is a possibility that corrosion will occur and will be localized, with the result that failure occurs more readily than in the absence of the coating.

9. There should be no contact between pipe and wood or cork or other porous substance. This applies to the exposed ends of the pipe as well as to the middle of the floor. The ends of the pipe protruding from the floor should be accessible so they can be inspected frequently and kept coated with paint or other protective coatings.

10. There is only a minor possibility that corrosion may be due to acid leachings from the cement.

11. The floor should not be poured over a cinder fill and cinders should not be used as aggregate.

12. Finally, the brine solution should be treated and tested at frequent intervals to see that the proper concentration of the reagents are present. The brine should be maintained slightly alkaline; the *pH* of the solution should be 8 or slightly above. Sodium bichromate should be added and concentration maintained in accordance with the recommendations of the American Society of Refrigeration Engineers. The advantage of this treatment is that it tends to reduce the corrosiveness of calcium chloride solutions, and in case of any leaks it would tend to protect the pipes.

13. The piping should be arranged so that when the system is shut down the pipes remain entirely filled with brine and are not exposed internally to air.

The following is an excerpt from "Refrigerating Data Book," published by The American Society of Refrigerating Engineers, Second Edition, 1934-35, page 404 on Corrosion in Refrigerating Plants.

. . . . . The utilization of chromic acid and the dichromates for corrosion inhibition has been based upon a foundation of laboratory and field work extending back for more than a hundred years. This treatment did not, however, come into general use in the refrigerating industries until this material had been collated and further experimental work undertaken by the above mentioned committee, formed in 1925. The committee recommended the use of sodium dichromate ($Na_2Cr_2O_7$-$2H_2O$) in preference to the direct use of the neutral chromate because of the price differential between the two. Neutral sodium chromate has since been placed on

the market and may be used for brine treatment in cases where it is not desirable to vary the acidity (pH value) of the brine under treatment . . . . . The committee recommended the use of the following quantities of sodium dichromate: 125 lb. per 1,000 ft.^{3} of calcium chloride ($CaCl_2$) brine; 200 lb. per 1,000 ft.^{3} of sodium chloride (NaCl) brine.

The use of caustic soda to convert the dichromate to the neutral chromate was also recommended, 27 lb. per 100 lb. of dichromate being suggested for neutral brines, with lesser quantities where the brine was originally alkaline. The necessity for close control of the alkalinity of the brine was stressed in this report and the maintenance of a slight alkalinity (pH 8.5) suggested. Subsequent experience has led many operators to prefer more nearly neutral brine solutions. . . . . .

In addition to accurately controlling the pH values of the brine it is necessary to maintain the proper concentration of sodium chromate or sodium dichromate if the corrosion prevention is to be effective. Analyses should be made of the brine every year or two in order to determine the actual chromate concentration present, and corrections may be based on the chemical analysis. If it is not feasible to have

QUANTITIES OF SODIUM DICHROMATE TO MAINTAIN INITIAL CONCENTRATION

Calcium Chloride Brine	
Specific Gravity	Lb. of Sodium Dichromate per 100 lb. $CaCl_2$(73%)
1.160	.695
1.169	.656
1.179	.621
1.188	.587
1.198	.556
1.208	.528
1.218	.502
1.229	.478
1.239	.455
1.250	.435

the analysis made, sodium chromate may be added to the brine when salt or calcium chloride is added for strengthening it. The recommended proportions should be observed if this method is adopted, and the ultimate density of the brine must also be taken into consideration if reasonably accurate control of the chromate concentration is to be achieved by this method.

The original report of the Corrosion Committee mentioned the possibility of chrome poisoning in connection with the use of sodium dichromate brine treatment. Experience in the ice industry has shown that this is a remote possibility where a reasonable degree of cleanliness is maintained. The freedom from difficulties of this nature is probably attributable to the fact that the brine when properly treated contains very small quantities of sodium chromate. Contact with concentrated solutions of sodium dichromate, or the other chromates, should be avoided, and care must be exercised when handling the strong chromate solutions to avoid the possibility of contact and injury to the skin. As ordinarily used, sodium dichromate is a bright orange colored granule. It should be dissolved in warm water, and the resulting solution poured into the brine in the proper proportions. The direct application of granular sodium dichromate to the brine is not available as it does not go into solution readily in cold brine. . . . . .

6. REMOVAL AND DISPOSITION OF OLD FLOOR

Only the upper slab of concrete in which the floor pipes are imbedded was removed. This contained no steel reinforcing rods and the six-foot wide sections were held together only by the brine pipes.

FIG. 6—REMOVING OLD FLOOR SLAB

To remove this floor slab, cuts approximately 16 in. wide and at right angles to the pipe were made with pneumatic hammers to divide the slab into sections of a size to permit handling, and to expose the brine pipes so they could be burned off. These sections, approximately 6 by 25 ft. and weighing about 3½ tons each were broken loose from the bed and turned over, finished side down, by the use of a large truck (Fig. 6). The sections were then pulled outside the building and the concrete shattered off the pipes with a pile-driver hammer suspended from the boom of a crawler crane.

The junk sale value of the salvaged pipe practically offset the cost of removing and disposing of the old floor.

7. PRELIMINARY WORK

After the sand and wood sleepers were removed, the surface of the concrete subfloor was thoroughly cleaned with brooms and brushes and elevation checked at frequent intervals with an engineer's level. All cracks and holes were filled with cement grout and projections were chiseled off so that subfloor was not only level but practically smooth.

8. THE NEW FLOOR

The design of the new floor is patented and is known as the Carpenter Monolithic Concrete Floating Floor. It was designed and developed by M. R. Carpenter, Cleveland, Ohio, who has for some thirty years specialized in designing and installing refrigerating equipment, and particularly skating rinks.

The floor, as its name implies, is monolithic, the concrete being placed and finished in one continuous operation without expansion, contraction or construction joints. The sand cushion and wood

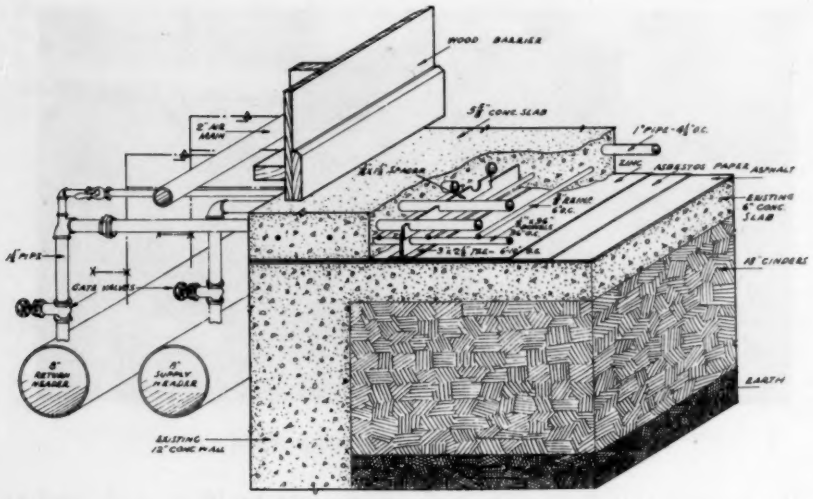

FIG. 7—DETAIL OF NEW FLOOR

sleepers were entirely omitted. The slab was increased in thickness to the former depth of the old floor plus the sand cushion, an average of 5⅝ in. (Fig. 7).

9. ASBESTOS PAPER AND ZINC BASE

The subfloor was first given a coat of concrete primer and then covered with a layer of 12 lb. unsaturated asbestos paper laid in hot asphalt. Extreme care was observed not to get asphalt or other foreign material on the surface of the asbestos. Where spots did occur, they were dusted over with talcum powder, the intention being to keep the surface thoroughly clean and smooth and reduce friction between the two slabs.

Over the asbestos paper was laid a layer of No. 5 sheet zinc. The sheets were laid with staggered joints, lapped one inch, and spot-soldered together to hold them in place (Fig. 8). The surfaces of contact between the zinc and the asbestos paper were carefully guarded to preserve their smoothness.

10. PIPE SUPPORTS

To support the brine pipe coils and to keep them to grade, rolled tee sections 2½ x 3 in. weighing 6.1 lb. per foot were used. These supports were placed directly on the zinc and spaced 6 ft. 10 in. on centers. These supports were set end on end and spliced together with bolted fish plates. Holes through the stems of the supports were drilled every 36 in. and through these ½ in. round dowel bars 36 in. long were inserted to bond together the adjacent sections of concrete

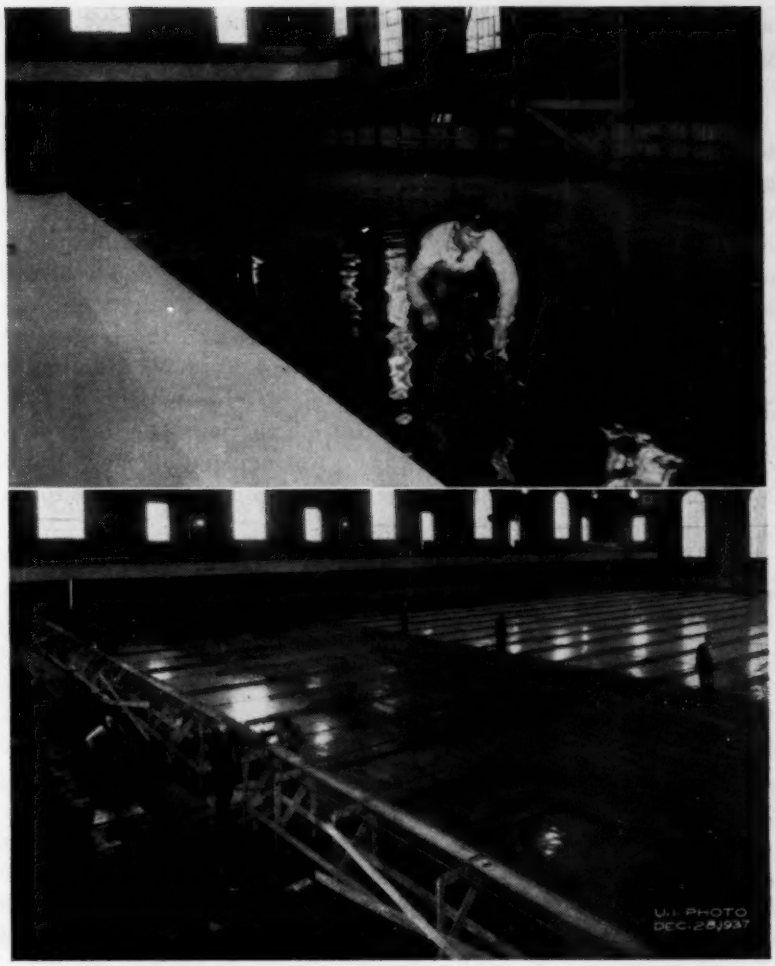

FIG. 8—SHEET-ZINK BETWEEN SUB FLOOR AND NEW SLAB
FIG. 9—WELDING PIPE

divided by these supports. Steel plates $\frac{1}{16}$ x $1\frac{1}{2}$ in. were spot welded in vertical position along the top edge of the tee support. This plate was notched every $4\frac{1}{2}$ inches to fit the outside diameter of the pipe coils and served to space and hold them accurately.

11. STEEL REINFORCEMENT

Steel reinforcing rods were spaced 8 in. apart at right angles to and underneath the pipe coils to which they were wired. One-half inch

round rods were used throughout the central portion of the rink and the size decreased to ⅜ in. round at each end.

12. PIPE WORK

The coils were constructed of standard weight 1-in. black steel butt-weld pipe furnished in 42 ft. lengths without mid-welds and with plain ends beveled for welding. The return bends were fabricated from 1-in., extra heavy weight, seamless, black steel pipe.

A temporary welding bench was erected across the full width of the floor upon which three lengths of pipe were laid, end on end for welding. The sections of pipe adjoining the main header tunnel were threaded and equipped with fittings to connect to the header pipe. Three lengths of pipe and the return bend were placed upon the bench, the abutting ends were securely clamped in contact and three welders set to work simultaneously to weld the joints as the pipe was slowly rotated (Fig. 9). A return bend was omitted on each alternate 126-ft. pipe section. The return bend was so constructed as to have one leg 10 and the other 19 in. long.

After the bench-welding operation, the assembled pipe was laid on the pipe supports and lined up in the spacers. The straight pipe without the return bend was connected to a reducing elbow of the supply header while the pipe with the return bend was assembled with screwed union to the return header. To complete the hairpin coils, a weld was made on the floor connecting the section with the return bend to its companion pipe. This work of welding and placing pipes was carried on progressively from one end of the floor to the other, the men carrying the pipes over those already laid.

The elevation of the pipes at numerous points over the rink was observed, and the highest point found was assumed to be the grade elevation of all the pipes. The pipes below this elevation were lifted to grade and, where necessary, wedges were placed temporarily under the pipe supports. Specially designed jacks were used to lift the tee supports and the superimposed pipes up to grade. Cement grout was then placed under the supports to hold the pipe system to grade.

In this new floor system, there are 516 sections of pipe forming 258 hairpin coils. The total length of pipe is 64,059 ft.—total number of welds, 1548.

13. SUPPLY AND RETURN MAIN HEADER CONNECTIONS

The original design provided for a continuous air vent from the connection to the supply header and a pet cock for bleeding out air on the connection to the return header. This made it necessary to bleed out air by hand each time the system was filled with brine or when coils had become air-locked. The flow of brine was reversed by

changing the supply and return connections to the main headers so that the circulating pump forced the air out ahead of the brine through the continuous vent to the atmosphere. The pet cocks were no longer needed and were removed.

Since the brine supply tank was below the level of the floor pipes, each time the circulating pumps were shut down, the entire system drained back into the tank. This caused considerable trouble and annoyance as the system was continually becoming air-bound, retarding circulation of the brine. This condition was corrected by installing a vertical loop in the return main, extending it above the floor level.

14. TESTING THE PIPE SYSTEM

The next operation was to test the pipes, welded joints, and screwed connections for leaks. The system was entirely filled with warm water under pressure of 60 p.s.i. The normal operating pressure on the system is about 4 lb. Warm water was used to prevent condensation on the outside of the pipes. The pressure was maintained for 48 hours and during this time all welded joints were hammered severely to test the welds. Two leaks were detected, one in a welded joint and one in a cracked fitting.

After the pipes were tested, the exposed pipes (i.e. those extending beyond the concrete slab), the fittings and headers were given a priming coat of red lead and a finish coat of aluminum paint.

15. THE CONCRETE FLOOR SLAB

The floor design provided for a concrete floor slab 5⅝ in. thick. It was constructed without permanent joints of any kind. The expansion and contraction of the slab was assumed to have no detrimental effect since the slab is not restrained at the edges and is free to slide or "float" on the comparatively frictionless base. The computed increase in length, due to change in temperature from 15° F. to 95° F. is about 1¼ in.

Screeds for carrying the straightedges and strike-off boards were designed of steel tee iron sections welded to chairs which were notched to fit the floor pipes upon which they rested. The upper edge of the tee section was precisely 1¼ in. above the top of the pipes.

Plank runways upon which the concrete was hauled were supported directly upon the floor pipes. The main runway extended along the center of the rink from the south door toward the north end of the floor, where it connected with a cross runway extending east and west immediately south of the strip of concrete to be poured. One section of loose planks only 4 ft. wide was used to connect this transverse runway with the place where the concrete was to be deposited.

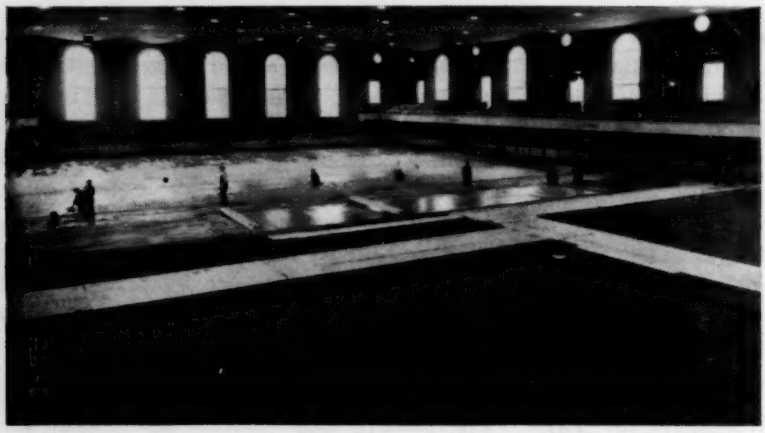

FIG. 10—PLACING THE CONCRETE

Placing of the concrete was started at the northwest corner of the rink and progressed toward the east (Fig. 10). The first strip was 14 ft.-4 in. wide; 15 other strips were each 12 ft. wide. As one strip was finished, the next strip south was started at the west side, and the work proceeded in this order until the entire floor was finished.

Trucks hauling the concrete from the mixing plant were backed up to the doorway and to the south end of the main runway, and the material was discharged directly into two-wheeled buggies of 6 cu. ft. capacity and then wheeled to the point of construction. To facilitate dumping the trucks without waste, delay or rehandling, the trucks were equipped with lever-operated tail-gate cut-off valves.

The concrete was well puddled around the floor pipes, pipe supports, and reinforcing steel, to insure the elimination of air pockets or voids and to preserve the uniform consistency of the concrete.

The surface was struck off twice in the usual manner by using a heavy straightedge, reaching from screed to screed, operated by two men. The concrete was then covered with dry burlap, and over the burlap a mixture of sand and cement was scattered to absorb excess moisture and to facilitate uniform hardening of the concrete.

When sufficiently hard to support a man's weight, (usually in about two hours) burlap with sand and cement mixture was removed and the concrete surfaced with rotary float machines equipped with tamping attachments. This process leveled out all irregularity, compacted the concrete mass, and worked up sufficient mortar to finish the surface. Following the machine work, hand finishers troweled the surface four times, the last time by using the trowel on edge, when the concrete was well set up.

As soon as the floor was sufficiently hardened, water was sprinkled over it and the surface kept wet down for ten days. During the placing of the concrete and throughout the curing period, water at 70° F. was circulated through the floor pipes to maintain a uniform temperature. The air temperature in the room was maintained at 65° F. during the same period.

16. CONCRETE AGGREGATE

The concrete was made from Red Ball "high early strength" or rapid hardening cement, torpedo sand and crushed limestone. The stone was graded 95 per cent passing a ½ in. sieve, 90 per cent passing a ⅜ in. sieve, and 15 per cent passing a ¼ in. sieve. The mixture of these materials for maximum density and minimum shrinkage was determined from laboratory analysis made at the Talbot Laboratory. The specifications resulting from these tests provided for a ratio of 1:2.56:3.05 or, stated in lb. per cu. ft.: 21.9 lb cement; 56.0 lb. sand; and 66.7 lb. stone.

The concrete was mixed in a 14-S electrically driven batch mixer at the central mixing plant. The cement was measured by the sack, and the sand and rock weighed through a batch hopper. The mixer was equipped with an accurate water control device and with a batch timer and counter to insure uniform mixture. The concrete was mixed with warm water and its temperature upon delivery at the job averaged 70° F.

17. DELIVERY TO THE RINK

Two trucks hauling four batches or about 1.91 cu. yds. each were used to deliver the concrete to the rink. A third truck was held in readiness and a standby mixer was provided to insure a continuous uninterrupted operation. The distance from the mixing plant to the rink was exactly one mile.

The first truck load of concrete arrived at the rink at 6:35 a. m., January 10, 1938. The construction work progressed at uniform speed without delay or accident and the last truck load was delivered at 3:30 a. m. January 12, or just forty-five hours after the first one.

The total net volume of concrete in the floor was 411.3 cu. yds. A goal was set and the work organized to construct the floor at the rate of 10 cu. yds. per hour. The rate actually attained was 9.8 cu. yds. per hour. The work was done by three shifts of men and was stopped only at lunch time.

18. TESTS

Observations were made after completion of the floor to verify the elevations, trueness and smoothness of the surface. Approximately 75 readings were taken at regular intervals over the entire floor and

the variation found was only plus or minus ⅛ in., and the average thickness of concrete over the pipes 1.234 in. as compared with the 1.250 in. specified.

Slump tests of the concrete were made frequently as it was being placed in the floor. The average of these tests was a slump of 4.3 in.

Samples of concrete were also taken from the trucks at intervals upon delivery to the rink and 6-in. by 12-in. test cylinders made. These were stored for 7 days at the rink alongside the floor, and then taken to the Talbot Laboratory where they were stored in air at 70° F. until tested. The average compressive strength of these specimens is given in Table 3.

TABLE 3—COMPRESSION TESTS OF 6 BY 12 IN. CONCRETE CYLINDERS
Made from concrete being place in University Skating Ring on January 10 and 11, 1938

Batch No.	Cyl. No.	Date Day	Made Hour	Slump of Concrete in.	Compression Strength—lb. per sq. in.		
					Age 7 Days	Age 14 Days	Age 28 Days
1	a	Jan. 10	7 A.M.	3.0	4070	4360	4780
	b				3980	4690	5179
2	a	Jan. 10	1 P.M.	4.2	3830	4920	4910
	b				4070	4945	5150
3	a	Jan. 10	7:30 P.M.	5.1	3835	4625	5450
	b				4070	4980	4970
4	a	Jan. 11	7 A.M.	6.2	3040	4110	5100
	b				3465	4400	5110
5	a	Jan. 11	1 P.M.	4.5	3685	4395	5140
					4020	4465	5500
6	a	Jan. 11	7:30 P.M.	2.8	3420	4320	4960
					3230	....	4250
Average				4.3	3730	4560	5040

A new refrigerating brine was made up consisting of a solution of calcium chloride gaged to a density of 1.250 sp. gr. and treated with sodium dichromate as recommended by the American Society of Refrigerating Engineers.

The new brine was first pumped through the pipe system on January 28, 1938. The temperature of the floor was reduced at the rate of 1° F. per hour, and on Jan. 30, ice was made on the new rink floor. The rink was officially reopened for public use on Feb. 1.

The writer gratefully acknowledges the services, valuable assistance and cooperation rendered by Dr. H. F. Johnstone, Prof. of Chem. Engr., John O. Kraehenbuehl, Prof. of Elect. Eng., Frank E. Richart, Research Prof. of Eng. Materials, and James Elmo Smith of the Physical Plant Dept. Staff.

Discussion, to close in February, 1940 JOURNAL, *should reach A. C. I. Secretary in triplicate by Dec. 1, 1939.*

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Discussion of a paper by John Doak

Reconstruction of the Ice Skating Rink at the University of Illinois*

BY ROY L. PECK†

In Mr. Doak's description of the failure of the original floor system he states that the actual cause of the first failure is not known, and that after the first leak developed the failure was progressive and rapid. Subsequent removal of the sections of the old floor disclosed several facts about the failure, which he has enumerated and explained more or less reasonably, but only in view of the obvious causes of the failure uncovered by the inspection of the old system.

I feel that this investigation missed the real reason or reasons why this failure occurred, and wish to point out in this discussion some facts which had been brought out prior to the failure of this floor, in the failure and investigation of another rink floor, built under the same system. This prior failure was studied by several engineers at the time it failed, and much was gained, the primary causes being similar to that of the University rink floor. This was the rink at the Chicago Stadium, replaced in 1934. The writer of this discussion acted in a consulting way as to the causes of failure there, and helped in the design of the system which was put in after removal of the old floor. The new system was designed to prevent like failures in the future, to be vastly more economical in freezing time under the conditions encountered there and at other large stadia or arenas, and to be economical in first cost.

The primary cause of failure was determined to have been damaging effects of stresses in the circulating pipe system and its encasing concrete, produced by rapid, frequent and extreme variations in

*JOURNAL Amer. Concrete Inst., Sept. 1939; *Proceedings*, Vol. 36, p. 1.
†Chicago.

temperature, to which a floor of this use is subjected throughout its life. This cause and other contributing causes are considered:

(1)—The temperature variation in the pipe itself is a maximum, and therefore the expansion and contraction of this portion of the system is greatest. There is a decided, measureable time-lag between the movement of the pipe under temperature change, and similar but unequal movement in the surrounding concrete, which produces undue stresses in both media, unless free longitudinal movement of the pipe under temperature change, in the encasing concrete, is provided for in the design. Such provision cannot be accomplished if the pipe is bonded to the surrounding concrete, to act as a reinforcing of such slab, or if the pipe is longitudinally constrained in any manner from free movement. The excessive stresses placed in the pipe system by this unequal movement may crack the pipe or break the joint, or may be transferred to the encasing concrete slab, which will tend to crack under such stresses. This damage is greatest as cold brine is forced into the pipe system after normal temperatures have existed, or as warm brine is introduced therein after freezing is finished. It is accumulative over long periods, and results in the leakage of the brine carried by the pipes, or of the water on the floor, through the encasing concrete down to the lower sections of the floor containing insulating materials, which in turn suffer the loss of their normal function by this liquid impregnation. The outside surfaces of all pipe should be coated with a film of separating material which serves two purposes. It permanently prevents the formation of a bond between the pipes and the encasing concrete and it coats the pipes with a water-resisting or moisture-resisting film, preventing corrosion of the pipes due to water electrolytic action. This film should be applied in a liquid condition, just prior to the placing of the concrete slab encasing the pipe system.

(2)—Another contributing cause of failure was creation of definite electric potentials between the pipe, carrying brine as the electrolyte, and the dissimilar metallic supports upon which the pipe system rested. Failures here are of a "punched hole" type, permitting leakage of brine with consequent damage as described in (1). Eliminating all metal-to-metal contact with the pipe itself is necessary to prevent the formation of electro-thermal-couples and electrolytic action in the presence of moisture. The use of concrete pre-cast supports, notched to receive the pipe, and made of the same mix of concrete as the encasing concrete slab, prevents electrolytic action, maintains the pipe in correct alignment and spacing and at a proper uniform level. The dimensions of these supports are such with respect to the load carried

that a relatively large area is offered, preventing punching of the membrane covering the insulating materials below the slab.

(3)—Free movement of that portion of the pipe system in the "hair-pin" return bends is highly necessary to reduce undue stress and consequent failure in these bends of the pipe and the encasing concrete, along this line of the floor. It is remembered that among the first failures of the University rink this type was noticed. Precast concrete boxes, of a shape to receive the bend of the pipe, and filled with heavy grease, were provided, to be subsequently encased in the floor slab concrete.

(4)—Speed in freezing any floor of this type, uniformly and equally at all points, is highly desirable. It was found that in the old floor system there were lines of slower frozen water existed during freezing. These points or lines were caused by partially air-bound pipe, occasioned by projections on the inner surface of the pipe, which in turn formed gas pockets within the pipe at the top level, preventing uniform passage of heat to and from the individual pipe line affected. Electric-resistance butt-welding of the pipe prevented the formation of any projections inside the pipe, and the introduction of any material foreign to the nature of the pipe, thus preventing the formation of thermal-couples and consequent electrolytic action and corrosion of pipe. Such welding also permitted free longitudinal movement of the pipe, no projections being formed on the outer surface of the pipe, which in effect would tend to anchor the pipe in the encasing concrete, causing undue stress in both media.

(5)—Failure of the waterproofed membrane covering the insulating media below, was occasioned by the punching of this membrane by the effect of the weight of the whole slab on separate supports of too-little area. Consequent impregnation of the cork layer practically eliminated its function. This layer of cork should be positioned as close as possible under the concrete slab encasing the pipes to reduce the heat loss downward into supporting material which should be upward to the surface, to effect fast freezing. The use of large area precast concrete supports accomplished the elimination of any failure by punching of the waterproofing membrane above the cork.

Summing up, Mr. Doak contends that failure in the brine pipes was due to external corrosion. I contend that the reason for the corrosion was due to leaks of brine through cracks or holes in the pipe from electrolytic action, but that these openings were first occasioned by the stresses in the pipe and concrete by the time-lag of movement between pipe and concrete, which point has evidently been completely missed by the author. Had free longitudinal move-

ment been provided for, such stresses would not have provided the first failure of which the investigation did not determine the true cause. Basically, the construction was such as to rapidly accelerate the effects of this first leak of brine or water. Other contentions he makes I concur in, those of not using a sand cushion, wood supports, steel turnings in the encasing slab, cinder aggregate, etc.

AUTHOR'S CLOSURE

Authorities all agree that there are numerous causes of corrosion. The causes of corrosion as set forth in the article were only those found after careful consideration to have had some bearing on skating rink failures.

It is well known that the stressing of steel does have some "damaging effects" as mentioned by Mr. Peck and under certain conditions is a possible source of corrosion. No evidence of this source, however, could be produced as a possible cause of the skating rink failure observed by the writer. In fact, in all failures observed the most serious corrosion took place where there was no bond between concrete and steel and the pipes were free to move.

The actual cause of the first failure at the University rink is not definitely known. The original corrosion was localized at the center of one end of the rink where moisture could seep through the wood blocks from the brass strips to the pipe as mentioned paragraph 3, page 7. When the floor was opened for repairs, corrosion was so prominent that the actual cause could not be determined. It could have been, and probably was, due to galvanic action as mentioned, although it is also possible that there were small pin holes in the pipe or faulty welds.

The writer is not familiar with the investigation of the cause of failure in the Chicago Stadium, and will therefore not dispute Mr. Peck's theory that failure was due to "damaging effects of stresses in the circulating pipe system and its encasing concrete, etc."

"Rapid, frequent, and extreme variations in temperature, etc." of the floor system may have been characteristic of the operation of the Chicago Stadium. However, it was not true at the University or several other similar rinks where ice is maintained throughout the season at more or less constant temperature.

Mr. Peck's statement that "there is a *decided* measureable time lag between the movement of the pipe under temperature change and similar but unequal movement in the surrounding concrete, etc.," is no doubt correct in theory but some explanation should be given as to how *decided* the change is and how it is measured. Mr. Peck's

theory that the bond between pipe and concrete should be broken intentionally is definitely erroneous and misses entirely one very pertinent source of corrosion because in breaking the bond there will be a slight air space formed around the pipe that will entrain free oxygen that will accelerate any corrosive condition.

The writer agrees with Mr. Peck that one of the contributing causes of failure is due to galvanic action from the use of dissimilar metals in the presence of moisture. This point is clearly set forth in the article, page 7, paragraph 3; page 8, item 3; and page 10, item 7.

Galvanic action from use of dissimilar metals could not have been entirely the cause of all rink failures. Brass valves and steel pipe were commonly used together in the majority of the rinks examined, and corrosion was not apparent inside or outside the pipe adjacent to the valves.

The electric potential between steel pipe and steel support causing the so called "punched hole" type of failure is questionable. Where such a failure has occurred, it is more than likely due to aerated water entrained in the voids of porous concrete around pipe supports which was characteristic of the original University rink and others that were examined.

Mr. Peck's theory that free movement must be provided for the "hair-pin" return bends is in error and from information available, is without foundation. The writer has supervised all repairs to the University rink and from personal observation there have been no failures in the bends as mentioned by Mr. Peck. Thorough investigation of other major rink failures did not reveal this as even a possible source of failure. Some rinks observed were designed with supply and return headers on opposite sides of the floor without return bends with the possibility of free longitudinal movement of the pipe encased in concrete. Failure was equally as severe and early as in the rinks designed with return bends.

Mr. Peck's theory as to cause of air bound pipes was no doubt hastily drawn and without thorough consideration of all facts available. The University rink had trouble with air binding in the pipes but this was not caused by projections inside the pipe. The cause of the trouble as outlined on page 16, last paragraph of the article, was eliminated. The new rink has been in operation for two seasons without trouble from air binding. It is true that projections inside the pipe are not desirable and should be eliminated, although not at the sacrifice of strength in the joints.

Some of the rinks observed employed screwed pipe with ends reamed and hydraulic couplings so that the projections in the pipe could not have been in these instances responsible for subsequent air binding and faulty circulation of brine. Neither could dissimilar metals of joint materials have caused failure since both pipe and couplings were of the same material.

Covering the pipe with a protective coating should prevent corrosion. This method of protection has one serious practical disadvantage in that a defect or damage to the covering will accelerate local corrosion and cause early failure.

Mr. Peck's reference to punching of the water proof membrane over cork insulation is irrelevant as concerns this article, as the University rink was not insulated. Speed in freezing the floor was not an important factor and the insulation was therefore eliminated.

In his summation, Mr. Peck contends that the corrosion was due to leaks of brine through cracked pipe or holes in pipe from electrolytic action. This is one of the several possible causes of failure as previously mentioned and is referred to in the original paper page 8, paragraph 3, items 2 and 3. In the University rink and other rinks examined there was no evidence of failure due to stresses in the pipe.

Since Mr. Peck so definitely attributes the cause of failure to stresses in pipe, the writer would like to know how the stresses are determined, and how they are related to the resistance in bond stress between the pipe and the concrete.

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Design Coefficients for Building Frames*

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SYNOPSIS

Modern building codes require analysis of building frames to be based on the principles of continuity. Many methods which have been presented in the literature on this subject are too laborious for the commercial design office. The intent of this paper is to present a short, easily applied, readily understood method which will give all of the accuracy necessary to comply with commercial and practical requirements. Ordinarily the designer is confronted with the problem of design knowing only the required loading and the center to center distances. His first fundamental assumption must be the determination of approximate stiffnesses. In the accompanying paper the tables have been so laid out that the designer can make this difficult assumption with all the accuracy that a final analysis will need.

The American Concrete Institute Building Code[1] requires:

All members shall be designed to resist at all sections the maximum bending moments and shears produced by dead load, live load and wind load, as determined by the principle of continuity (Sec. 701).

Further, the code makes provision for the following assumptions:

(a) Consideration may be limited to combinations of dead load on all spans with full live load on two adjacent spans and with full live load on alternate spans (Sec. 702).

(b) The far ends of columns above and below the floor under consideration may be considered fixed. (See 702)

Tables 1 and 2 have been prepared to comply with the above provisions of the code. Extreme accuracy has not been the all important consideration. It is realized that designers have a limited amount

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[1]Building Regulations for Reinforced Concrete (A. C. I. 501-36-T) tentatively adopted 1936

of time in which to prepare their designs and that their fee does not permit analysis requiring a design of accuracy not consistent with the limitations of present day knowledge of the physical constants involved. The moment of inertia, modulus of elasticity, and effective length, over a working range, are difficult if not impossible to evaluate exactly. Uncertainties of load and its distribution, as well as uncertainty of internal stress conditions, discourage overly refined and complex analysis.

What is desired by designers is a method easily and quickly applied, sufficiently accurate to give a well balanced and economical design. Use of arbitrary coefficients left much to be desired since the effect of variables involved could not be so simply expressed.

The following two tables permit a quick and accurate solution of moments needed for design in a great majority of building frames. Cases where supports offer little restraint and beam stiffnesses vary greatly are not covered by the tables.

Examples illustrate the make-up of Tables 1 and 2. Table 1 is for ideal frames, each of equal spans and constant column to beam stiffness ratios and having equal uniform loads of various ratios of live to dead load. The loads are placed to produce maximum end moments and maximum or minimum midspan moments.

Table 2 gives correction factors to be applied to coefficients of Table 1 where span lengths and loading vary from the ideal.

The procedure is very similar to the use of arbitrary moment coefficients with which all designers are familiar, yet a high degree of accuracy is obtained, by one additional step, which will comply with the "principle of continuity" as required by the A. C. I. Code.

EXPLANATION OF TABLES

Table 1

The assumptions used in computing Table 1 are:

1. Equal span lengths and uniform load.
2. Constant beam stiffnesses.
3. Various ratios of live to dead load and total column stiffness (ΣK_c) to stiffness of a single beam (K_b).
4. Columns fixed at story above and below floor under consideration.
5. Arrangement of live load to give maximum or minimum moments as tabulated.

Moment distribution was used in computing the values shown in this table and the values are accurate for the assumptions listed.

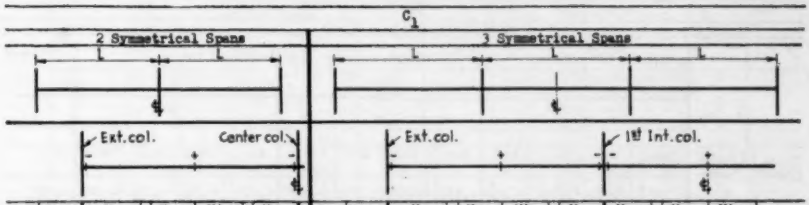
TABLE 1—MOMENT COEFFICIENTS—C_1

Uniform Load—Equal Spans and Beam Stiffnesses. Columns Fixed at Far Ends.

$M = C_1 WL$ or $M = C_1 (w_{Dead} + w_{Live}) L^2$

where: M = Moment in foot pounds, or foot kips.
L = Span in feet
W = Total load on span in pounds, or kips.

C_1



W live / W dead	K col. / K beam	Max. −	Max. +	Min. +	Max. −	W live / W dead	K col. / K beam	Max. −	Max. +	Min. +	Max. −	Max. −	Max. +	Min. +
	0	0	+.063	+.063	−.125		0	0	+.075	+.075	−.100	−.100	+.025	+.025
	1	−.042	+.052	+.052	−.104		1	−.044	+.055	+.055	−.096	−.088	+.037	+.037
0	2	−.055	+.049	+.049	−.098	0	2	−.058	+.049	+.049	−.093	−.085	+.040	+.040
	4	−.066	+.046	+.046	−.092		4	−.067	+.046	+.046	−.090	−.084	+.041	+.041
	8	−.074	+.044	+.044	−.088		8	−.074	+.044	+.044	−.087	−.084	+.041	+.041
	Inf.	−.083	+.042	+.042	−.083		Inf.	−.083	+.042	+.042	−.083	−.083	+.042	+.042
	0	0	+.075	+.031	−.125		0	0	+.083	+.042	−.106	−.106	+.042	0
	1	−.045	+.056	+.031	−.104		1	−.047	+.058	+.033	−.098	−.092	+.044	+.017
0.5	2	−.058	+.051	+.030	−.098	0.5	2	−.060	+.052	+.031	−.094	−.089	+.044	+.022
	4	−.069	+.047	+.029	−.092		4	−.069	+.048	+.029	−.091	−.086	+.044	+.025
	8	−.075	+.045	+.028	−.088		8	−.075	+.045	+.029	−.088	−.085	+.043	+.026
	Inf.	−.083	+.042	+.028	−.083		Inf.	−.083	+.042	+.028	−.083	.083	+.042	+.028
	0	0	+.078	+.016	−.125		0	0	+.088	+.025	−.108	−.108	+.050	−.013
	1	−.046	+.058	+.020	−.104		1	−.048	+.060	+.022	−.099	−.094	+.048	+.008
1	2	−.060	+.054	+.021	−.098	1	2	−.061	+.053	+.021	−.095	−.091	+.047	+.013
	4	−.070	+.048	+.021	−.092		4	−.070	+.048	+.021	−.091	−.088	+.045	+.016
	8	−.076	+.045	+.021	−.088		8	−.076	+.045	+.021	−.088	−.085	+.044	+.018
	Inf.	−.083	+.042	+.021	−.083		Inf.	−.083	+.042	+.021	−.083	−.083	+.042	+.021
	0	0	+.083	0	−.125		0	0	+.092	+.008	−.111	−.111	+.058	−.025
	1	−.048	+.060	+.009	−.104		1	−.050	+.062	+.011	−.100	−.096	+.052	−.002
2	2	−.061	+.053	+.011	−.098	2	2	−.062	+.054	+.012	−.095	−.093	+.049	+.003
	4	−.071	+.049	+.012	−.092		4	−.071	+.049	+.013	−.091	−.089	+.047	+.008
	8	−.077	+.046	+.013	−.088		8	−.077	+.046	+.013	−.088	−.086	+.045	+.011
	Inf.	−.083	+.042	+.014	−.083		Inf.	−.083	+.042	+.014	−.083	−.083	+.042	+.014
	0	0	+.086	−.008	−.125		0	0	+.094	0	−.113	−.113	+.063	−.031
	1	−.049	+.061	+.004	−.104		1	−.050	+.063	+.005	−.101	−.097	+.054	−.007
3	2	−.062	+.054	+.007	−.098	3	2	−.063	+.055	+.007	−.096	−.094	+.050	−.001
	4	−.071	+.049	+.008	−.092		4	−.071	+.049	+.008	−.091	−.090	+.047	+.004
	8	−.077	+.046	+.009	−.088		8	−.077	+.046	+.009	−.088	−.087	+.045	+.007
	Inf.	−.083	+.042	+.010	−.083		Inf.	−.083	+.042	+.010	−.083	−.083	+.042	+.010

Table 2

Values in Table 2 are corrections which may be applied to values of Table 1 when span lengths and/or uniform load intensities vary from span to span. These corrections are accurate in all cases where the stiffness of the long or short beam is the same as that of adjacent beams. An accuracy sufficient for design is also obtained when the stiffness is different from that of adjacent beams, as will be shown in examples.

Table 2 may be explained as follows: Consider an interior beam AB having a length aL and loaded with a total uniform unit load of w_{TaL} as compared to the condition assumed in Table 1, i.e., a span L and a total uniform unit load of w_{TL}. If the difference in fixed end moments of span AB under the two conditions is distributed through

TABLE 1 (continued)—MOMENT COEFFICIENTS—C_1

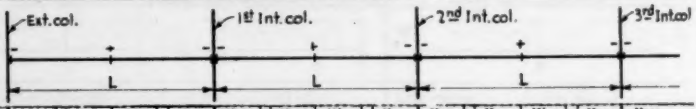
Uniform Load—Equal Spans and Beam Stiffnesses. Columns Fixed at Far Ends.

$M = C_1WL$ or $M = C_1 (w_{Dead} + w_{Live})L^2$

where: M = Moment in foot pounds, or foot kips.
L = Span in feet
W = Total load on span in pounds, or kips.

C_1

4 or More Symetrical Spans



$\frac{W_{Live}}{W_{Dead}}$	$\frac{\Sigma K_{col.}}{K_{beam}}$	Max. −	Max. +	Min. +	Max. −	Max. −	Max. +	Min. +	Max. −	Max. −	Max. +	Min. +	Max. −	Max. −
0	0	0	+.072	+.072	-.106	-.106	+.034	+.034	-.077	-.077	+.044	+.044	-.085	-.085
	1	-.044	+.054	+.054	-.098	-.090	+.039	+.039	-.081	-.082	+.042	+.042	-.084	-.084
	2	-.057	+.050	+.050	-.094	-.087	+.041	+.041	-.082	-.083	+.042	+.042	-.083	-.083
	4	-.067	+.046	+.046	-.090	-.085	+.042	+.042	-.083	-.083	+.042	+.042	-.083	-.083
	8	-.074	+.044	+.044	-.087	-.084	+.042	+.042	-.083	-.083	+.042	+.042	-.083	-.083
	Inf.	-.083	+.042	+.042	-.083	-.083	+.042	+.042	-.083	-.083	+.042	+.042	-.083	-.083
0.5	0	0	+.081	+.039	-.110	-.110	+.049	+.007	-.088	-.088	+.057	+.016	-.094	-.094
	1	-.046	+.058	+.033	-.100	-.094	+.047	+.019	-.087	-.088	+.049	+.021	-.089	-.089
	2	-.060	+.052	+.031	-.095	-.090	+.045	+.022	-.086	-.087	+.047	+.023	-.087	-.087
	4	-.069	+.048	+.029	-.091	-.087	+.044	+.026	-.085	-.086	+.044	+.025	-.086	-.086
	8	-.076	+.045	+.029	-.088	-.085	+.043	+.026	-.085	-.085	+.043	+.026	-.085	-.085
	Inf.	-.083	+.042	+.028	-.083	-.083	+.042	+.028	-.083	-.083	+.042	+.028	-.083	-.083
1	0	0	+.085	+.023	-.113	-.113	+.056	-.006	-.094	-.094	+.063	+.002	-.099	-.099
	1	-.048	+.060	+.022	-.100	-.096	+.050	+.009	-.090	-.091	+.052	+.011	-.092	-.092
	2	-.061	+.053	+.021	-.095	-.091	+.048	+.013	-.088	-.089	+.049	+.014	-.089	-.089
	4	-.070	+.048	+.021	-.091	-.088	+.046	+.018	-.087	-.087	+.046	+.017	-.087	-.087
	8	-.076	+.045	+.021	-.088	-.086	+.044	+.018	-.085	-.085	+.044	+.018	-.085	-.085
	Inf.	-.083	+.042	+.021	-.083	-.083	+.042	+.021	-.083	-.083	+.042	+.021	-.083	-.083
2	0	0	+.090	+.007	-.115	-.115	+.064	-.019	-.099	-.099	+.070	-.011	-.104	-.104
	1	-.050	+.062	+.011	-.101	-.098	+.054	-.001	-.093	-.094	+.056	0	-.095	-.095
	2	-.063	+.054	+.013	-.096	-.093	+.050	+.004	-.090	-.091	+.051	+.005	-.091	-.091
	4	-.071	+.049	+.013	-.091	-.089	+.047	+.011	-.088	-.088	+.047	+.008	-.088	-.088
	8	-.077	+.046	+.013	-.088	-.086	+.045	+.011	-.086	-.086	+.045	+.011	-.086	-.086
	Inf.	-.083	+.042	+.014	-.083	-.083	+.042	+.014	-.083	-.083	+.042	+.014	-.083	-.083
3	0	0	+.092	-.002	-.116	-.116	+.068	-.026	-.102	-.102	+.073	-.018	-.106	-.106
	1	-.050	+.063	+.005	-.101	-.099	+.055	-.006	-.094	-.095	+.057	-.005	-.096	-.096
	2	-.064	+.055	+.007	-.096	-.094	+.051	0	-.091	-.092	+.052	0	-.092	-.092
	4	-.072	+.049	+.008	-.091	-.090	+.048	+.007	-.089	-.089	+.048	+.004	-.089	-.089
	8	-.077	+.046	+.009	-.088	-.087	+.045	+.007	-.086	-.087	+.045	+.007	-.087	-.087
	Inf.	-.083	+.042	+.010	-.083	-.083	+.042	+.010	-.083	-.083	+.042	+.010	-.083	-.083

the frame, the final moments will represent the corrections due to change in load and span of AB. The difference in fixed end moments at joint A is equal to $\frac{1}{12}(w_{TaL}a^2L^2 - w_{TL}L^2)$ while the difference in fixed moments at B is $-\frac{1}{12}(w_{TaL}a^2L^2 - w_{TL}L^2)$. We may therefore compute the correction to Table 1 by distributing the coefficients, $\frac{1}{12}$, and multiplying the results by $(w_{TaL}a^2L^2 - w_{TL}L^2)$. Let the $\frac{1}{12}$ factors after distribution be designated by V, then the correction factor to be applied to Table 1 is

$V(w_{TaL}a^2L^2 - w_{TL}L^2)$ which may be written as

$$V\left(\frac{w_{TaL}}{w_{TL}}a^2 - 1\right)w_{TL}L^2.$$

The end moment correction coefficients as listed in Table 2 are the product of V (determined accurately) and values of $\frac{w_{TaL}}{w_{TL}}a^2 - 1$.

TABLE 2—CORRECTIONS TO MOMENT COEFFICIENTS OF TABLE 1

(Use when spans or loads vary)

$M = CWL$ or $M = C\,(w_{Dead} + w_{Live})\,L^2$

$C = C_1$ (from Table 1) + C_2 (from Table 2)

$\frac{\Sigma K_{col.}}{K_{beam}}$	$a\sqrt{\frac{w_{TaL}}{w_{TL}}}$	Interior Spans – C_2 at L					$\frac{\Sigma K_{col.}}{K_{beam}}$	$a\sqrt{\frac{w_{TaL}}{w_{TL}}}$	Exterior Spans – C_2 at L			
		Adj. span	Long or short span corrections			Adj. span			Adj. span	Long or short span corrections		
	1.4	-.050	-.050	+.069	-.050	-.050		1.4	-.061	-.061	+.090	0
	1.3	-.036	-.036	+.050	-.036	-.036		1.3	-.044	-.044	+.065	0
	1.2	-.023	-.023	+.032	-.023	-.023		1.2	-.028	-.028	+.041	0
	1.1	-.011	-.011	+.015	-.011	-.011		1.1	-.013	-.013	+.020	0
0	1.0	0	0	0	0	0	0	1.0	0	0	0	0
	.9	+.010	+.010	-.014	+.010	+.010		.9	+.012	+.012	-.018	0
	.8	+.019	+.019	-.026	+.019	+.019		.8	+.023	+.023	-.034	0
	.7	+.027	+.027	-.037	+.027	+.027		.7	+.032	+.032	-.048	0
	.6	+.034	+.034	-.046	+.034	+.034		.6	+.041	+.041	-.060	0
	1.4	-.030	-.063	+.057	-.063	-.030		1.4	-.032	-.068	+.061	-.049
	1.3	-.022	-.046	+.041	-.046	-.022		1.3	-.023	-.049	+.044	-.035
	1.2	-.014	-.029	+.026	-.029	-.014		1.2	-.015	-.031	+.028	-.022
	1.1	-.007	-.014	+.012	-.014	-.007		1.1	-.007	-.015	+.013	-.011
1	1.0	0	0	0	0	0	1	1.0	0	0	0	0
	.9	+.006	+.013	-.011	+.013	+.006		.9	+.006	+.013	-.012	+.010
	.8	+.011	+.024	-.021	+.024	+.011		.8	+.012	+.026	-.023	+.018
	.7	+.016	+.034	-.030	+.034	+.016		.7	+.017	+.036	-.033	+.026
	.6	+.020	+.042	-.038	+.042	+.020		.6	+.021	+.045	-.041	+.033
	1.4	-.022	-.068	+.052	-.068	-.022		1.4	-.023	-.071	+.054	-.061
	1.3	-.016	-.049	+.037	-.049	-.016		1.3	-.016	-.051	+.039	-.044
	1.2	-.010	-.031	+.024	-.031	-.010		1.2	-.010	-.033	+.025	-.028
	1.1	-.005	-.015	+.011	-.015	-.005		1.1	-.005	-.016	+.012	-.013
2	1.0	0	0	0	0	0	2	1.0	0	0	0	0
	.9	+.004	+.014	-.010	+.014	+.004		.9	+.005	+.014	-.011	+.012
	.8	+.008	+.026	-.019	+.026	+.008		.8	+.009	+.027	-.020	+.023
	.7	+.011	+.036	-.028	+.036	+.011		.7	+.012	+.038	-.029	+.033
	.6	+.014	+.045	-.035	+.045	+.014		.6	+.015	+.047	-.036	+.041
	1.4	-.014	-.073	+.047	-.073	-.014		1.4	-.014	-.074	+.048	-.070
	1.3	-.010	-.052	+.034	-.052	-.010		1.3	-.010	-.053	+.035	-.050
	1.2	-.006	-.033	+.022	-.033	-.006		1.2	-.007	-.034	+.022	-.032
	1.1	-.003	-.016	+.010	-.016	-.003		1.1	-.003	-.016	+.011	-.015
4	1.0	0	0	0	0	0	4	1.0	0	0	0	0
	.9	+.003	+.014	-.009	+.014	+.003		.9	+.003	+.015	-.010	+.014
	.8	+.005	+.027	-.018	+.027	+.005		.8	+.005	+.028	-.018	+.026
	.7	+.007	+.039	-.025	+.039	+.007		.7	+.008	+.039	-.026	+.037
	.6	+.009	+.049	-.031	+.049	+.009		.6	+.009	+.049	-.032	+.047
	1.4	-.008	-.076	+.044	-.076	-.008		1.4	-.008	-.076	+.044	-.075
	1.3	-.006	-.055	+.032	-.055	-.006		1.3	-.006	-.055	+.032	-.054
	1.2	-.004	-.035	+.020	-.035	-.004		1.2	-.004	-.035	+.020	-.034
	1.1	-.002	-.017	+.010	-.017	-.002		1.1	-.002	-.017	+.010	-.016
8	1.0	0	0	0	0	0	8	1.0	0	0	0	0
	.9	+.002	+.015	-.009	+.015	+.002		.9	+.002	+.015	-.009	+.015
	.8	+.003	+.028	-.017	+.028	+.003		.8	+.003	+.028	-.017	+.028
	.7	+.004	+.040	-.024	+.040	+.004		.7	+.004	+.040	-.024	+.040
	.6	+.005	+.051	-.029	+.051	+.005		.6	+.005	+.051	-.030	+.050
	1.4	0	-.080	+.040	-.080	0		1.4	0	-.080	+.040	-.080
	1.3	0	-.057	+.029	-.057	0		1.3	0	-.057	+.029	-.057
	1.2	0	-.037	+.018	-.037	0		1.2	0	-.037	+.018	-.037
In-	1.1	0	-.017	+.009	-.017	0	In-	1.1	0	-.017	+.009	-.017
fin-	1.0	0	0	0	0	0	fin-	1.0	0	0	0	0
ity	.9	0	+.016	-.008	+.016	0	ity	.9	0	+.016	-.008	+.016
	.8	0	+.030	-.015	+.030	0		.8	0	+.030	-.015	+.030
	.7	0	+.042	-.021	+.042	0		.7	0	+.042	-.021	+.042
	.6	0	+.053	-.027	+.053	0		.6	0	+.053	-.027	+.053

To simplify the use of the table $a\sqrt{\frac{w_{TaL}}{w_{TL}}}$ has been listed to represent $\left(\frac{w_{TaL}}{w_{TL}}\,a^2 - 1\right)$.

The corrections for the maximum midspan moment coefficients listed in Table 1 for interior spans were obtained from the expression

$$\text{Correction} = \tfrac{1}{8}\left[w_{TaL}a^2L^2 - w_{TL}L^2\right] - \text{V}\left[w_{TaL}a^2L^2 - w_{TL}L^2\right]$$

$$= \left[\tfrac{1}{8} - \text{V}\right]\left[\frac{w_{TaL}}{w_{TL}}\,a^2 - 1\right] w_{TL}L^2$$

When the end coefficients are different, as in the case of an exterior span, then the maximum positive moment correction as listed in Table 2 is obtained by averaging the end moments and subtracting from the simple beam moment, or

$$\text{Correction} = \tfrac{1}{8}\left[w_{TaL}a^2L^2 - w_{TL}L^2\right] - \frac{V_A + V_B}{2}\left[w_{TaL}a^2L^2 - w_{TL}L^2\right]$$

$$= \left[\tfrac{1}{8} - \frac{V_A + V_B}{2}\right]\left[\frac{w_{TaL}}{w_{TL}}\,a^2 - 1\right] w_{TL}L^2$$

The correction for minimum midspan moment requires a slightly different treatment because the span in question carries dead load only when the minimum condition exists.

Let the span in question have a length of aL as before and carry a unit uniform dead load of w_{DaL}, instead of a length of L and loaded with a unit uniform dead load of w_{DL}. By the same reasoning as used for total uniform load we may say that the difference in end moment correction equals

$$V\,(w_{DaL}a^2L^2 - w_{DL}L^2)$$

This may be written as $V\left[w_{DaL}a^2L^2 - w_{DL}L^2\right]\dfrac{w_{TL}L^2}{w_{TL}L^2}$

$$\text{or}\left[V\right]\left[\frac{w_{DaL}}{w_{DL}}\,a^2 - 1\right]\left[\frac{w_{DL}}{w_{TL}}\right] w_{TL}L^2.$$

Therefore, correction for minimum midspan moment is

$$\left[\tfrac{1}{8} - V\right]\left[\frac{w_{DaL}}{w_{DL}}\,a^2 - 1\right]\left[\frac{w_{DL}}{w_{TL}}\right] w_{TL}L^2.$$

Let $a\sqrt{\dfrac{w_{DaL}}{w_{DL}}}$ represent $\left[\dfrac{w_{DaL}}{w_{DL}}a_2 - 1\right]$, enter Table 2 with values of $a\sqrt{\dfrac{w_{DaL}}{w_{DL}}}$ instead of $a\sqrt{\dfrac{w_{TaL}}{w_{TL}}}$ and multiply midspan correction coefficient by $\dfrac{w_{DL}}{w_{TL}}$. The final coefficient is applied to midspan moment coefficient of Table 1.

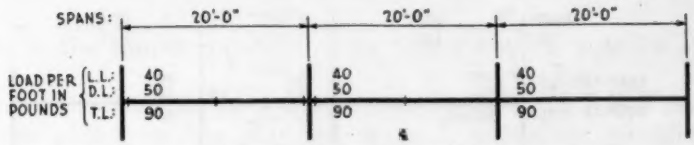
USE OF THE TABLES

Example No. 1

The calculation of maximum moments in a 3-span frame is given in Example No. 1. Equal span lengths and constant ratio of live to dead

3-SPAN SYMMETRICAL FRAME

ASSUME: DIMENSIONS AND LOADING AS SHOWN
COLUMN STIFFNESS = (K_{COL}) = BEAM STIFFNESS (K_{BEAM})



(1) ASSUME LL=D.L. ENTER TABLE I (FOR 3-SPANS) WITH $\frac{LL}{DL}=1$ AND $\frac{\Sigma K_{COL.}}{K_{BEAM}}=2$

	EXTERIOR SPAN			INTERIOR SPAN	
TABLE I:	-.061	+.053	-.095	-.091	+.047
MULTIPLIER: $w_{TL}L^2=(40+50)\,20^2=36{,}000$ FT. LB.					
MAX. MOMENTS (FT. LB.)	-2200	+1910	-3420	-3280	+1690

(2) ACTUAL LIVE TO DEAD LOAD RATIO $\frac{LL}{DL}=\frac{40}{50}=0.80$ AND $\frac{\Sigma K_{COL.}}{K_{BEAM}}=2$

TABLE I:	-.061	+.053	-.095	-.090	+.046
MULTIPLIER: 36,000 FT. LB.					
MAX. MOMENTS (FT. LB.)	-2200	+1910	-3420	-3240	+1660

EXAMPLE NO. 1

load permit the use of Table 1 for precise moments directly. Assume, in this case, that the position of the frame in a structure is such that column stiffnesses (or support stiffnesses) are approximately equal to beam stiffness (or slab stiffness, as the case may be). A considerable error in this assumption will have small effect on the maximum moments, as will be shown in later examples.

Using a live to dead load ratio of one $\left(\frac{L.L.}{D.L.}=\frac{40}{50}=1,\ \text{approx.}\right)$ and total column stiffness to beam stiffness of two $\left(\frac{\Sigma K_{col}}{K_{beam}}=2\right)$, moment coefficients from Table 1 are tabulated across the sketched frame. Note that Table 1 is separated into three parts, one for two-span; one for three-span, and one for four or more span frames. Since only maximum moments are desired, the coefficients for minimum positive moments in Table 1 are ignored.

Each recorded coefficient is multiplied by total uniform load per foot times square of span length in feet. The multiplier, $w_{TL}L^2$, equals $(40 + 50)\ 20^2 = 36{,}000$ ft.lb. Maximum moments are recorded below the multiplier, with minus signs indicating tension in top face and plus signs, tension in bottom face of beams.

3-SPAN SYMMETRICAL FRAME

SPANS:		20'-0"			14'-0"		20'-0"
aL:		1.0 L			0.7 L		1.0 L
LOAD PER FOOT IN POUNDS	L.L.:	40			100		40
	D.L.:	50			50		50
	T.L.:	90 = w_{TL}			150 = w_{TaL}		90
$a\sqrt{\frac{w_{TaL}}{w_{TL}}}$:		1.0			$0.7\sqrt{\frac{150}{90}} = 0.90$		1.0
					℄		
$\frac{L.L.}{D.L.} = 1$, $\frac{\Sigma K_{COL.}}{K_{BEAM}} = 2$	TABLE I:	-.061	+.053	-.095	-.091	+.047	← SAME VALUES AS EXAMPLE 1 (1)
$a\sqrt{\frac{w_{TaL}}{w_{TL}}} = 0.9$, $\frac{\Sigma K_{COL.}}{K_{BEAM}} = 2$	TABLE II: (INTERIOR SPAN)			+.004	+.014	-.010	
FINAL MOMENT COEFF.(SUM)		-.061	+.053	-.091	-.077	+.037	
MULTIPLIER: $w_{TL}L^2 = (40+50)\,20^2 = 36{,}000$ FT. LB.							
MAXIMUM MOMENTS: (FT. LB.)		-2200	+1910	-3280	-2770	+1330	
CHECK ON ABOVE:							
MAXIMUM MOMENTS: (DETERMINED PRECISELY ASSUMING CONSTANT BEAM STIFFNESS)		-2230	+1980	-3260	-2760	+1300	
MAXIMUM MOMENTS: (DETERMINED PRECISELY ASSUMING CONSTANT BEAM DEPTH)		-2230	+1980	-3270	-2850	+1330	

EXAMPLE NO. 2

In part (2) of the example the actual live to dead load ratio is used in selection of coefficients from Table 1. This involves interpolation between coefficients for load ratios of 0.5 and 1. It is seen that coefficients are not sensitive to small changes in live to dead load ratio. In general, it is sufficiently accurate to use values of load ratio in the table without interpolation.

Example No. 2

The frame of Example No. 2 is the same as in Example No. 1 except that the interior span and live load are 14′0″ and 100 pounds per ft. instead of 20′0″ and 40 pounds per ft., respectively. Coefficients from Table 1 are recorded as before, the change in interior span and loading being ignored at first.

Correction coefficients from Table 2 are now introduced because of the short span. Table 2 is separated into two parts, one for interior and one for exterior spans, and is based on various values of $\frac{\Sigma K_{col}}{K_{beam}}$

and $a\sqrt{\frac{w_{aL}}{w_L}}$. The last expression takes in the effect of variable span and variable load. Using the left span and loads as standard, calculation of the proper $a\sqrt{\frac{w_{TaL}}{w_{TL}}}$ value (0.90) for the interior span is made. Correction coefficients are taken from Table 2 and recorded in their places on the sketched frame. Maximum moments are computed by adding algebraically the coefficients of Tables 1 and 2, and multiplying through by $w_{TL}L^2$.

4-SPAN CONTINUOUS FRAME, AS SHOWN

SPANS:	18'-0"			21'-6"			15'-0"			17'-0"		
aL:	1.0L			1.2L			0.83L			0.94L		
LOAD PER FOOT IN POUNDS L.L.:	300			300			300			300		
D.L.:	100			100			100			100		
T.L.:	400			400			400			400		
$a\sqrt{\frac{w_{TaL}}{w_{TL}}}$	1.0 (STANDARD SPAN)			1.2			0.83			0.94		
(1) $K_{COL.} = 2K_{BEAM}$												
$\frac{\Sigma K_{COL.}}{K_{BEAM}} = 4$	EXTERIOR SPAN			1ST INTERIOR SPAN			1ST INTERIOR SPAN			EXTERIOR SPAN		
$\frac{L.L.}{D.L.} = 3$ TABLE I:	-.072	+.049	-.091	-.090	+.048	-.089	-.089	+.048	-.090	-.091	+.049	-.072
TABLE II:			-.006	-.033	+.022	-.033 +.004	-.006 +.023	-.015	+.023 +.002	+.004 +.009	-.006	+.008
FINAL MOM. COEFF.:	-.072	+.049	-.097	-.123	+.070	-.118	-.072	+.033	-.065	-.078	+.043	-.064
MULTIPLIER: $(100+300)18^2 = 129,600$ FT. LB.												
MAXIMUM MOMENTS: (FT. LB.)	-9400	+6400	-12600	-16000	+9100	-15300	-9300	+4300	-8400	-10100	+5600	-8300
PRECISE MOMENTS: (CONSTANT BEAM STIFF.)	-9200	+6400	-12700	-15800	+8700	-15000	-8900	+3700	-8400	-10100	+5700	-8400
(2) $K_{COL.} = K_{BEAM}$												
$\frac{\Sigma K_{COL.}}{K_{BEAM}} = 2$ TABLE I:	-.064	+.055	-.096	-.094	+.051	-.091	-.091	+.051	-.094	-.096	+.055	-.064
$\frac{L.L.}{D.L.} = 3$ TABLE II:			-.010	-.031	+.024	-.031 +.007	-.010 -.022	-.016	+.003 +.022	+.008 +.007	-.007	+.007
FINAL MOM. COEFF.:	-.064	+.055	-.106	-.125	+.075	-.115	-.079	+.035	-.069	-.081	+.048	-.057
MAXIMUM MOMENTS: (FT. LB.)	-8300	+7100	-13700	-16200	+9700	-14900	-10200	+4500	-8900	-10500	+6200	-7400
(3) $K_{COL.} = 0$ (POINT SUPPORTS)												
MAXIMUM MOMENTS: (COMPUTATIONS OMITTED)	0	+11900	-18000	-18000	+13000	-14100	-14100	+6000	-12000	-12000	+10500	0

EXAMPLE NO. 3

As a check on the accuracy of the moments the frame was analyzed precisely—first, assuming a constant beam stiffness in the frame and, second, assuming constant beam depths. Which assumption, if either, is correct depends on the actual design of the beams after maximum moment calculation.

Example No. 3

A frame of four unequal spans is considered in this example. Using the left span as a standard the "aL" spans are calculated. Since the live and dead loads are constant across the frame, the values of $a\sqrt{\frac{w_{TaL}}{w_{TL}}}$ are equal to the respective values of a. With these values of a, corrections can be obtained from Table 2 by interpolation. Maximum moments may now be calculated if the stiffness ratio is given.

Experience in design of building frames and knowledge of the position of the frame in the structure will allow a rough estimate to be made of ratio of column stiffness to beam stiffness of the standard span. Maximum moments are given in the example for three assumptions of stiffness ratio, to indicate the possible variation. There should be no question as to which assumption is most nearly correct in a given case. Intermediate computations have been omitted in part (3) of the example so that the designer can check the results independently, by use of Tables 1 and 2.

Example No. 4

The frame of this example has been chosen to illustrate the use of the tables for unequal spans and loads.

After choosing the left span as the standard, and estimating a stiffness ratio of 2, the various values of a and $a\sqrt{\frac{w_{TaL}}{w_{TL}}}$ are computed. Coefficients from Table 1 are recorded directly and coefficients from Table 2 are calculated by simple interpolation, and recorded in the proper columns. A check on the maximum moments by precise analyses shows the close agreement.

The occurrence of a short span between two long spans makes it advisable to check for reversal of midspan moment. The *minimum* positive midspan coefficient, first interior span, is taken from Table 1. Table 2 is entered with a value of $a\sqrt{\frac{w_{aL}}{w_L}}$ computed by using dead loads instead of total loads. The new coefficient ($-.035$) is multiplied by the ratio of standard span dead load to standard span total load to obtain the correction coefficient. The sum of the coefficients, multiplied by the constant $w_{TL}L^2$, gives the minimum midspan moment

4-SPAN CONTINUOUS FRAME, AS SHOWN

ASSUME: $K_{COL.} = K_{BEAM}$ $\quad \frac{\Sigma K_{COL.}}{K_{BEAM}} = 2$

SPANS:-	20'-0"			12'-0"			22'-0"			21'-0"		
a L:-	1.0L			0.6L			1.1L			1.05L		
LOAD PER FOOT IN KIPS L.L.	3.0			4.5			2.5			2.8		
LOAD PER FOOT IN KIPS D.L.	1.5			1.5			1.5			1.5		
LOAD PER FOOT IN KIPS T.L.	4.5			6.0			4.0			4.3		
$a\sqrt{\frac{w_{TaL}}{w_{TL}}}$	1.00			$0.6\sqrt{\frac{6.0}{4.5}} = 0.69$			$1.1\sqrt{\frac{4.0}{4.5}} = 1.04$			$1.05\sqrt{\frac{4.3}{4.5}} = 1.02$		
$\frac{L.L.}{D.L.} = \frac{3.0}{1.5} = 2$												
$\frac{\Sigma K_{COL.}}{K_{BEAM}} = 2$	EXTERIOR SPAN			1ST INTERIOR SPAN			1ST INTERIOR SPAN			EXTERIOR SPAN		
TABLE I:	-.063	+.054	-.096	-.093	+.050	-.090	-.090	+.050	-.093	-.096	+.054	-.063
			+.011	+.036	-.028	+.036	+.011					
TABLE II:						-.002	-.006	+.004	-.006	-.002		
									-.001	-.003	+.002	-.003
FINAL MOM. COEFF.	-.063	+.054	-.085	-.057	+.022	-.056	-.085	+.054	-.100	-.101	+.056	-.066
MULTIPLIER: $w_{TL}L^2 = (3.0+1.5)20^2 = 1,800$												
FINAL MOMENTS (KIP FT.)	-113	+97	-153	-103	+40	-101	-153	+97	-180	-182	+101	-119
CHECK ON ABOVE:												
PRECISE MOMENTS (CONSTANT BEAM STIFF.)	-115	+102	-151	-101	+39	-98	-149	+99	-181	-184	+101	-116
PRECISE MOMENTS (CONSTANT DEPTH)	-113	+100	-156	-111	+41	-106	-153	+96	-180	-183	+101	-118

12 FT. BEAM MIN. MIDSPAN MOMENT: $a\sqrt{\frac{w_{DaL}}{w_{DL}}} = 0.6$

COEFFICIENT FROM TABLE II: -.035

COEFFICIENT FROM TABLE I: +.004

$-.035 \times \frac{w_{DL}}{w_{TL}} = -.035 \times \frac{1.5}{4.5} = -.0117$

SUM = -.0077

12 FT. BEAM MINIMUM MIDSPAN MOMENT = -.0077 (1800) = -14

(DETERMINED PRECISELY, M = -13)

EXAMPLE NO. 4

in the short span. The procedure just described is general for determination of minimum moment in any span.

Example No. 5

A frame having five unequal spans and various combinations of loads is presented to illustrate a more general use of the tables. Two new steps will be noted in the example. The first is in the selection of the $a\sqrt{\frac{w_{TaL}}{w_{TL}}}$ value for concentrated loads. Since Table 1 is based on uniform loads, it is necessary, when concentrated loads are considered, to enter Table 2 with the proper constant to represent the change in maximum moments produced by such loads.

The total concentrated load on a span divided by the span length gives an "equivalent" uniform load, p_{TaL}, to compare with the standard total uniform unit load, w_{TL}. If the equivalent load, p_{TaL}, is multiplied by the ratio of fixed end moment coefficients—concentrated load coefficient divided by uniform load coefficient ($\frac{1}{12}$)—then Table 2 can be used as in other examples.

The following expressions take the place of $a\sqrt{\frac{w_{TaL}}{w_{TL}}}$ for concentrated loads:

(1) A single concentration at midspan, $a\sqrt{\frac{1.5p_{TaL}}{w_{TL}}}$

(2) Two concentrations at third-points, $a\sqrt{\frac{1.33p_{TaL}}{w_{TL}}}$

(3) Three concentrations at quarter-points, $a\sqrt{\frac{1.25p_{TaL}}{w_{TL}}}$

In the example, Table 1 coefficients are recorded by using the standard span live to dead load ratio, and the proper stiffness ratio. Coefficients from Table 2 are based, of course, on the respective $a\sqrt{\frac{w_{TaL}}{w_{TL}}}$ values.

A third correction (marked with an asterisk) introduces the second new feature of the example. Midspan corrections for concentrated loads, as tabulated, are based in part on fixed end moment ratios, which are not constant for net moment at midspan. A third correction must be *added* to midspan coefficients for all spans having concentrated loads. It can be shown that these corrections are:

(1) For a single concentration at midspan, $0.042\left[a\sqrt{\frac{1.5p_{TaL}}{w_{TL}}}\right]^2$

(2) For two loads at third-points, 0

(3) For three loads at quarter-points, $0.008\left[a\sqrt{\frac{1.25p_{TaL}}{w_{TL}}}\right]^2$

Note that in each expression the portion within the bracket has already been computed for use in Table 2, and the third correction is therefore easily obtained.

Procedures for Spans Having Both Concentrated and Uniform Loads

It is not uncommon for spans having concentrated loads that some uniform load must also be considered. The effect of the load combina-

5-SPAN CONTINUOUS FRAME
VARIABLE SPANS AND LOADING

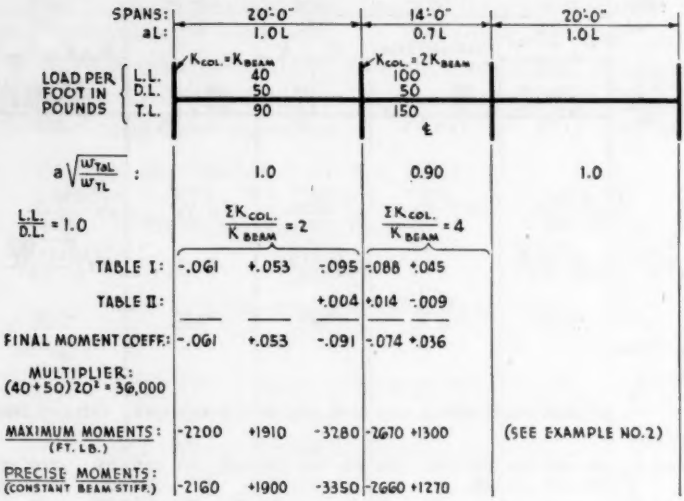
SPANS:		21'-0"			20'-0"		16'-0"			22'-0"			24'-0"				
aL:		1.05 L			1.0L		0.8L			1.1L			1.2 L				
		7'	7'	7'	(STANDARD SPAN)		8'	8'					6'	6'	6'	6'	
LOAD IN KIPS OR KIPS PER FOOT	L.L.:		10	10	1.0			19		0.9				5.5	5.5	5.5	
	D.L.:		4	4	0.5			8		0.4				3.0	3.0	3.0	
	T.L.:		28		1.5 K./FT.			27		1.3 K./FT.				25.5			
$a\sqrt{\frac{w_{TaL}}{w_{TL}}}$:		$a\sqrt{\frac{1.33p_{TaL}}{w_{TL}}}$			$1.0\sqrt{\frac{1.5}{1.5}}$		$a\sqrt{\frac{1.5p_{TaL}}{w_{TL}}}$			$1.1\sqrt{\frac{1.3}{1.5}}$			$a\sqrt{\frac{1.25p_{TaL}}{w_{TL}}}$				
		$+1.05\sqrt{\frac{1.33\times\frac{28}{21}}{1.5}}$					$+0.8\sqrt{\frac{1.5\times\frac{27}{16}}{1.5}}$						$+1.2\sqrt{\frac{1.25\times\frac{25.5}{24}}{1.5}}$				
		$=1.14$			$=1.00$		$=1.04$			$=1.02$			$=1.13$				
$K_{COL}=2K_{BEAM}$																	
$\frac{\Sigma K_{COL.}}{K_{BEAM}}=4$																	
		EXTERIOR SPAN			1ST INTERIOR SPAN		2ND INTERIOR SPAN			1ST INTERIOR SPAN			EXTERIOR SPAN				
$\frac{L_L}{D_L}=2$																	
TABLE I:		-.071	+.049	-.091	-.089	+.047	-.088	-.088	+.047	-.088	-.088	+.047	-.089	-.091	+.049	-.071	
TABLE II:		-.022	+.015	-.023	-.005								-.004	-.021	+.014	-.020	
				0	0	0	0			-.001	-.003	+.002	-.003	-.001			
							-.001	-.006	+.004	-.006	-.001						
3RD CORRECTION*			0*						+.045*						+.010*		
FINAL COEFF.:		-.093	+.064	-.114	-.094	+.047	+.089	-.094	+.096	-.095	-.092	+.049	-.096	-.113	+.073	-.091	
MULTIPLIER: (0.5+1.0)20²=600																	
MAX. MOMENT: (FT. KIPS)		-56	+38	-68	-56	+28	-53	-56	+58	-57	-55	+29	-58	-68	+44	-55	
PRECISE MOMENTS: (CONSTANT BEAM STIFF)		-56	+39	-69	-56	+28	-54	-56	+58	-57	-55	+29	-58	-68	+44	-55	
*3RD CORRECTION (TO MAX. MIDSPAN MOM. BECAUSE OF CONCENTRATED LOADS ON SPAN)		0 (NO CORRECTION FOR LOADS AT THIRD-POINTS)					$+0.042\left[a\sqrt{\frac{1.5p_{TaL}}{w_{TL}}}\right]^2$ $+0.042\times1.04^2$ $=+0.045$						$+0.008\left[a\sqrt{\frac{1.25p_{TaL}}{w_{TL}}}\right]^2$ $+0.008\times1.13^2$ $=+0.010$				

EXAMPLE NO. 5

tions may be included by one of two procedures, either of which is satisfactory. By the first, simply add in the total uniform load on the span to the loads of the concentrations and proceed in computation of values of $a\sqrt{\frac{Cp_{TaL}}{w_{TL}}}$ as in Example No. 5. Maximum moments determined in this way will be slightly larger than the theoretical moments, but usually by only insignificant amounts.

The second and more precise procedure is in the use of the modified expression, $a\sqrt{\frac{Cp_{TaL}+w_{TaL}}{w_{TL}}}$, instead of $a\sqrt{\frac{Cp_{TaL}}{w_{TL}}}$ used in Example No. 5. The factor C is, of course, 1.5, 1.33 or 1.25, depending on the number of concentrated loads on the span.

3-SPAN CONTINUOUS FRAME
VARIABLE SUPPORT STIFFNESSES


SPANS: 20'-0" 14'-0" 20'-0"
aL: 1.0L 0.7L 1.0L
$K_{COL.} = K_{BEAM}$
$K_{COL.} = 2K_{BEAM}$
LOAD PER FOOT IN POUNDS
L.L. 40 100
D.L. 50 50
T.L. 90 150
$a\sqrt{\frac{w_{TOL.}}{w_{TL}}}$: 1.0 0.90 1.0
$\frac{L.L.}{D.L.} = 1.0$
$\frac{\Sigma K_{COL.}}{K_{BEAM}} = 2$
$\frac{\Sigma K_{COL.}}{K_{BEAM}} = 4$
TABLE I: -.061 +.053 -.095 -.088 +.045
TABLE II: +.004 +.014 -.009
FINAL MOMENT COEFF.: -.061 +.053 -.091 -.074 +.036
MULTIPLIER: (40+50)20² = 36,000
MAXIMUM MOMENTS (FT. LB.): -2200 +1910 -3280 -2670 +1300 (SEE EXAMPLE NO.2)
PRECISE MOMENTS (CONSTANT BEAM STIFF.): -2160 +1900 -3350 -2660 +1270

EXAMPLE NO. 6

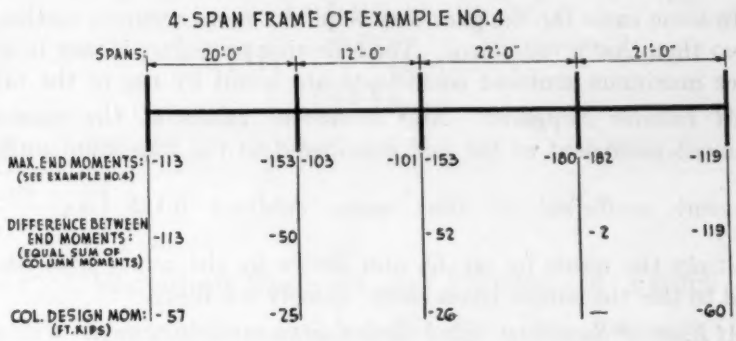
Example No. 6—Variable Column Stiffnesses

It has been emphasized that the actual stiffness ratios in a frame are practically indeterminate, and that a reasonable estimate of the ratios is satisfactory for use in the tables. This example illustrates the procedure for frames having relatively slender exterior columns compared to interior columns. Assume that the $\frac{\Sigma K_{col}}{K_{beam}}$ ratio is 2 for exterior columns and 4 for interior columns. A suggested procedure is to record all coefficients of exterior spans using the ratio 2, and the interior span or spans using the ratio 4.

A comparison of the maximum moments with those calculated in Example No. 2 shows the small change in moments from the different assumptions. Only in rather extreme cases is it considered justifiable to employ varying stiffness ratios across a frame.

Example No. 7—Column Moments

For the design of columns it is customary to use the maximum column load at any level, including live load on adjacent spans. If the effect of moment is to be included, the loading pattern should not be live load on alternate spans, but should be live load on the two spans adjacent to the column under consideration and live load also on alternate spans therefrom. This loading pattern is identical to the



EXAMPLE NO. 7

pattern for maximum negative moment at the beam ends adjacent to the column. *The column moment is then the difference between maximum end moments at the column, divided by 2.* If the column above the joint is different from the one below, to obtain each column moment take the difference in maximum end moments multiplied by a ratio equal to the stiffness of the column divided by the total stiffness of the two columns.

These column moments tend to produce tension above the joint and compression below the joint on the side of the larger beam moment.

The conventional loading pattern for column maximum moments places live load on alternate spans. Larger moments than those described above will be produced, but also less direct load on the columns. It is considered more logical, in general, to design columns using moments by the procedure first described, but the tables may also be used to calculate column moments by the latter procedure. Exterior column moments are the same, of course, by both load patterns.

Example No. 7 illustrates the calculation of column moments for the frame used in Example No. 4. Maximum end moments are copied from this example.

Shear Determination by the Tables

It is common practice in calculating end shear of *interior* beams in a frame to neglect the shear due to continuity—that is, the shear due to unequal end moments of the beam. The shear for design is assumed equal to one-half the total load on the span. The designer may wish to include the increased shear due to continuity at the interior support of *exterior* beams. It is estimated to be about 20 per cent of the simple beam shear. The shear for design in this case is then six-tenths (0.6) of the total load on the span.

In some cases the designer may require a more accurate method for shear than that given above. The following procedure is easy to apply after maximum moment coefficients are found by use of the tables.

At Interior Supports: Add *numerical* values of the maximum moment coefficient at the end considered to the maximum midspan moment coefficient of that span, subtract $0.125 \left[a\sqrt{\frac{w_{TaL}}{w_{TL}}}\right]^2$, multiply the result by $w_{TL}L^2$, and divide by the *actual* span length. Add to this the simple beam shear, namely $0.5\ W_{TaL}$.

At Exterior Supports: Find shear due to continuity as at an interior support, but multiply it by *two* before adding the simple beam shear.

These rules appear formidable, but are actually easy to apply. For example, consider the first interior column from the right end of the frame in Example No. 4.

The design shear at the left of the column is, using the final moment coefficients,

$$V_{Left} = (0.100+0.054-0.125\text{x}1.04^2)\ \frac{1800}{22} + 0.5\ (4.0\text{x}22.0)$$

$$= 2 + 44 = 46 \text{ kips.}$$

The shear at the right column (an exterior column) is,

$$V = 2\ (0.066+0.056-0.125\text{x}1.02^2)\ \frac{1800}{21} + 0.5\ (4.3\text{x}21.0)$$

$$= -1 + 45 = 44 \text{ kips.} \quad \text{(For design, use 45 kips).}$$

Discussion, to close in February, 1940 JOURNAL, should reach A. C. I. Secretary in triplicate by Dec. 1, 1939.

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Effect of Weight of Tampers and Number of Tamps on the Flexural Strength of Concrete Silo Staves*

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SYNOPSIS

502 concrete silo staves were made at a commercial plant for the tests reported. The transverse strengths were determined for 324 of these staves. The chief variables were the weight of tampers and the number of tamps per stave. The number of staves per sack, the aggregate grading, and the consistency were also varied. For the materials and methods of manufacture used—

(a) Increasing the weight of the tampers from 50 lb. to 75 lb., increased the transverse strength, the increase varying both with the number of tamps and the mix.

(b) The transverse strength increased with increase in number of tamps from 2 tamps to 6 tamps. Beyond 6 tamps the effect on the transverse strength was influenced also by the mix and by the weight of the tampers.

(c) In nearly all cases staves made with 2 tamps of the heavy tamper were stronger than staves made with 6 tamps of the normal one.

(d) The thickness of tamped staves varies considerably. Hence if the load per inch of width is to be used as a measure of quality, it should be corrected to correspond to a specified thickness.

*A report of progress of experiments conducted by the Engineering Experiment Station and the Agricultural Experiment Station of the University of Minnesota, the United States Department of Agriculture, and the Department of Conservation, State of Minnesota. Received by the Institute May 2, 1939.

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SCOPE OF INVESTIGATION

The investigation was planned originally to include the following variables:

Cement content—	7 staves per sack
	9 staves per sack
	11 staves per sack
Aggregate grading—	Fine
	Plant
	Coarse
Consistency—	Dry
	Plant
	Wet

Since flow is implied by the term consistency, its use in connection with tamped concretes is an incongruity. However, for lack of a better term, it is here used as for plastic concretes to distinguish between wet and dry mixes.

Standard portland cement was used for all mixes except one for which high early strength cement was used. The plant grading and the plant consistency were those used in commercial operation of the plant. Special care was taken so that at 6 tamps all staves would be of commercial quality as regards to straightness. As it turned out, all staves regardless of the number of tamps, were straight enough for commercial use. Hence no data in this report result from so-called "experimental" staves. All data shown are averages of tests of three staves, one from each of three batches.

AGGREGATE

The properties of the three sizes of aggregate used—pea gravel, medium sand, and fine sand—are given in Table 1.

TABLE 1—PROPERTIES OF AGGREGATE

	Pea gravel	Medium sand	Fine sand
Bulk specific gravities	2.65	2.63	2.59
Absorption, per cent by weight	0.83	0.65	0.84
Unit weights, pounds per cubic foot	104.5	111.0	97.1
Voids, ratio by volume	0.37	0.32	0.40
Mechanical analysis			

Sieve size	Per cents Retained		
	Pea gravel	Medium sand	Fine sand
½ inch	0		
¾ inch	2	0	
No. 4	91	3	
No. 8	99	19	
No. 16	100	38	0
No. 30		67	4
No. 50		93	42
No. 100		99.5	91

The combinations of the three aggregates were selected on the basis of sieve analyses made some months previous to this investigation.

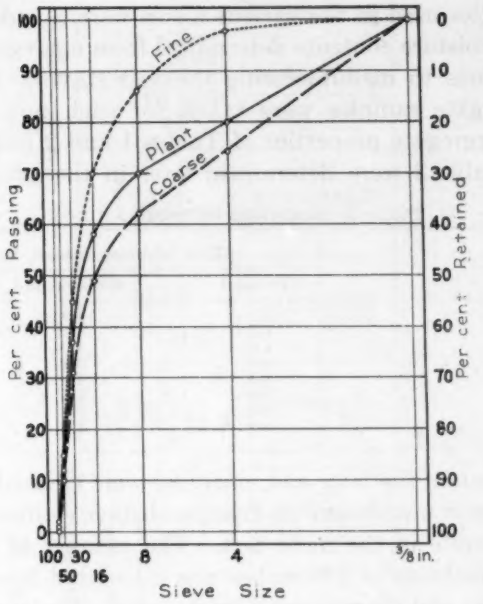

FIG. 1—MECHANICAL ANALYSIS OF COMBINED AGGREGATES

It was found later that the gradings had changed so that there was not as much difference in the properties of the combinations as had been expected. The proportions of each aggregate of Table 1 used in the combined aggregates and the properties of the combined aggregates determined later in the laboratory are given in Table 2. The mechanical analysis of the combined aggregates are plotted in Fig. 1.

TABLE 2—PROPERTIES OF COMBINED AGGREGATES

	Plant grading	Fine grading	Coarse grading
Per cent fine sand	10.5	11.0	9.75
Per cent medium sand	70.2	87.0	65.25
Per cent pea gravel	19.3	0	25.00
Bulk specific gravity	2.63	2.63	2.63
Absorption, per cent by weight	0.70	0.67	0.71
Unit weights, lb. per cubic foot	118.	112.	118.00
Voids, ratio by volume	0.28	.32	0.28

For the majority of mixes used in tamped concrete products, it is known that the volume of paste (water plus cement) is less than the volume of voids in the aggregate. Consequently, the assumption that the volume of concrete equalled the bulk volume of the aggregate appeared reasonable. This assumption permitted the advance calculation of the weights of cement and aggregate for each mix. The weight of cement in each sack was adjusted to exactly 94 lb.

The quantities used in the various mixes were calculated from unit weights and moisture contents determined from aggregates in the bins the day previous to manufacturing the test staves. During manufacture, aggregate samples were taken for each mix. From these samples the aggregate properties of Tables 1 and 2 and the moisture contents of Table 3 were determined later in the laboratory.

TABLE 3—MOISTURE DETERMINATIONS

Mix number	Total Moisture Content, Per Cents by Weight		
	Fine sand	Medium sand	Pea gravel
2	4.2	4.9	3.3
4	4.2	5.1	3.2
5	4.6	4.9	3.1
6	4.3	5.4	5.3
7	4.3	5.0	
1	5.6	5.3	3.0
3	4.5	5.0	3.4
11	4.9	3.9	3.4

After each batch, the floor and machines were cleaned up, the number of full staves counted, and the fractional stave estimated by placing the excess concrete in the mold box. The number of staves corresponding to a thickness of 2½ inches was calculated from the counted number of staves and the measured thicknesses. The number of staves per sack of cement calculated on the assumption that the volume of concrete equals the bulk volume of the aggregate and the counted number of staves 2½-in. thick per sack of cement are given in Table 4. Differences between the nominal number of staves per sack and the calculated number of staves per sack result from the use of the laboratory unit weights and of the moisture determinations pertaining to the various mixes.

TABLE 4—NUMBER OF STAVES PER SACK

Mix number	Calculated from bulk volume of aggregate	Counted (2½ inch thickness)	Ratio $\frac{\text{Counted}}{\text{Calculated}}$
1	6.83	7.19	1.05
2	8.75	9.14	1.04
3	10.75	11.06	1.03
4	8.75	9.06	1.03
5	8.75	9.13	1.04
6	8.82	9.50	1.08
7	8.95	8.87	0.99
11	10.80	10.93	1.01

Using the concrete volumes corresponding to the counted staves per sack of Table 4, the mix characteristics of Table 5 were calculated. In reading Table 5, it should be remembered that accuracy of its data depends on the count of the staves and the assumption that the volume of staves 2½ in. thick was 0.434 cu. ft. as well as upon the accuracy of the aggregate tests. Whatever inaccuracies exist would have the greatest effect on the air-voids.

TABLE 5—CONCRETE CHARACTERISTICS

Mix No.	No. of staves per sack	Consistency	Aggregate		Ratio of volume of paste to voids in aggregate	Absolute volume per unit volume		Water cement ratio gallons per sack	Aggregate cement ratio by volume	Voids, cement ratio by volume	Transverse strength ratios average for normal & heavy tamping at 6 tamps
			Nominal grading	Voids, ratio by volume		Voids v	Air voids va		a/c	v/c	
1	7.19	plant	plant	0.28	1.11	0.166	0.021	3.40	4.41	1.08	1.67
2	9.14	plant	plant	0.28	0.85	0.193	0.084	3.24	5.66	1.60	1.09
3	11.06	plant	plant	0.28	0.75	0.203	0.097	3.79	6.86	1.99	1.00
4	9.06	wet	plant	0.28	0.88	0.183	0.067	3.52	5.64	1.51	1.27
5	9.13	dry	plant	0.28	0.81	0.190	0.090	2.96	5.66	1.56	0.98
6	9.50	plant	coarse	0.28	0.95	0.216	0.087	3.98	5.74	1.86	1.19
7	8.87	plant	fine	0.32	0.79	0.195	0.065	3.72	5.50	1.56	1.14
11*	10.93	plant	plant	0.28	0.67	0.191	0.104	3.08	6.88	1.87	1.23

*High early strength cement

For all mixes except No. 1 the ratio of the volume of paste to the volume of voids in the aggregate was less than 1. Excepting mix 1, the total number of staves calculated by use of the assumption that the volume of concrete equals the bulk volume of the aggregate was 449.2 and the number of equivalent counted staves was 463.8. Thus on the average the volume of concrete was 1.03 times the bulk volume of the aggregate. For mix 1 the ratio of the sum of the absolute volumes of the ingredients to the bulk volume of the aggregate was 1.03. Consequently, the assumption used in calculating the quantities of the solid constituents was found to be correct enough for practical use.

QUANTITY OF WATER

No method was known by which the quantity of added water could be calculated prior to mixing. Consequently, the quantity of added water was determined at the mixer by the judgment of the plant superintendent for the first batch of each mix and then, if necessary, modified for succeeding batches from the appearance of the staves.

MIXING

Batches were mixed in a Blystone 25 cu. ft. mixer for 1 minute dry and for 3½ minutes after the addition of water. All ingredients were weighed. Each batch was divided between two tamping machines. Because of possible segregation of the concrete, the first two and last two staves from each batch were not included in the test specimens.

TAMPING MACHINES

One machine, an Anchor, was fitted with additional weights on the tamper feet. The other machine, a Stearns, specially built so as to be essentially similar to the Anchor was used without modification. For convenience the former will be referred to as the heavy machine and the latter as the normal or plant machine. The weights of tampers and heights of blow are given in Table 6.

TABLE 6—WEIGHT AND FALL OF TAMPERS

Machine		1	2	3	4	5	6	Total
Normal	Weight of tamper, lb.	52¾	49¼	49¼	49¾	49¾	52½	
	Height of fall, in.*	15⅜	15⅝	16	15¼	15¾	16⅛	
	Potential energy foot pounds	68.6	64.1	65.6	63.3	64.6	70.5	396.7
Heavy	Weight of tamper, lb.	75	74	75	76	75	76	
	Ht. of fall, in.*	15½	16	15¾	16	16⅜	15⅞	
	Potential energy foot pounds	96.9	98.6	98.5	101.2	102.1	99.9	597.2
Ratio—heavy to normal								1.48

*The height of fall was measured from top of mold box to the locking stop positions of the tampers. The rate of tamping was timed to be 7 blows per second or about 1 fall per tamper per second.

The usual procedure in the Waterloo plant is to use 6 blows of each tamper in molding followed by 4 blows of each tamper on the finishing plate. In this investigation the mold was filled, tamped, filled, and the finishing plate placed in position. The number of blows used per tamper in molding were as follows:

Normal machine.....	2	6	10	14	18	22	26
Heavy machine.....	2	6	10	14	18		

The third stave per machine was given 2 blows per tamper, the fourth 6 blows, and so on up to 26 for the normal machine and up to 18 for the heavy machine. In all cases 4 blows per tamper were used on the finishing plate.

The departures from a straight line 28 in. long were measured at both sides of all staves by means of a specially mounted Ames dial. The average departure for the staves of this investigation was 0.014 in. The range except for three measurements of 0.036, 0.032, and 0.038 was 0.001 to 0.031. The plant limitation on straightness was said to be a departure of $\frac{1}{32}$ inch (0.031 inches).

The plant superintendent, on inspecting the staves for appearance at the edges after curing, designated 265 as O. K., 53 as fair, and 6 as poor, out of a total of 324.

CURING

All staves were cured in the same moist room until tested. In addition to the use of steam, the staves were sprinkled with a hose twice a day. Because the curing room could not be kept out of plant use for a longer period, the staves were tested at an average age of 7 days.

TRANSVERSE TESTS

The staves were tested immediately after removal from the moist room in a cross-bending machine at the plant. The test procedure was according to the A. C. I. specification, submitted by Committee P-4, Jan. 20, 1926. Because it was thought that the relative strengths show the effects of the several variables better than the actual strengths only strength ratios are given.

Measurement of the thickness of the staves indicated considerable variation in this dimension. As the transverse strength of staves is expressed by the load per inch of width, and this quantity varies directly with the square of the depth, correction for a depth of 2.5 in. results in considerable modification of the load per inch of width. For example, the load per inch of width for a stave 2.75 in. thick made from the same quality concrete as a stave 2.50 in. thick would be 21 per cent greater than that for the 2.50 in. stave. Hence if the load per inch of width is to be a criterion of the quality of the concrete,

it must be reduced to correspond to a constant depth of stave. As 2.50 in. is the nominal depth for a stave, all strength data were reduced to correspond to this thickness. Obviously, the need for corrections would be eliminated if the modulus of rupture were used as the criterion of strength.

The effect of the number of blows of the tampers is shown in Fig. 2 in which the ratios of the transverse strengths to the strength at 6 tamps are plotted against the number of blows per tamper, using the open circle and dashed line for the plant tampers and the solid circle and full line for the heavy tampers. It will be seen that the effect varied considerably with the different mixes and in some cases, notably with mix 2, when the heavy instead of the plant tampers were used. For all mixes using the plant tampers, and for all but two mixes using heavy tampers, an increase in transverse strength resulted from increasing the number of tamps from 2 to 6.

In these tests when the number of tamps was increased above 6, it would appear that increased tamping first consolidated the concrete thus causing an increase in strength and that additional tamping beyond some number of blows instead of increasing the consolidation had a churning effect thus decreasing the strength. Such alternations may occur several times, hence the hills and valleys of the graphs of Fig. 2. From these tests it is evident that while increased tamping increases the strength up to some maximum, further increase in the number of blows may result in a decrease in strength and that the number of tamps required for maximum transverse strength varies with the proportions, the aggregate, and the machine used.

In the lower right hand corner of Fig. 2 data are shown from two other plants. The graphs marked B1 and B2 were obtained from data on staves made from concrete from the same batches. The B1 staves were made on a machine, having five tampers falling simultaneously. The B2 staves were made on a machine having 6 tampers falling in succession. The C graph is from the data presented by Miller[1], corrected to correspond to a 2.5 inch thickness.

In order to permit some comparison of the effect of the number of blows of the tampers to the effect of slight changes in the proportions, grading, or consistency, the ratios of the strengths at six blows of the tampers of the various mixes having nominally 7, 9, and 11 staves per sack to the strengths of mixes 1, 2, and 3 respectively have been plotted in Fig. 2 using a cross for plant tamping and a solid square for heavy tamping. For example, staves made from mix No. 4 using 6 heavy

[1]Miller, Dalton G., "Factors which influence the durability of concrete silo staves." JOURNAL Amer. Concrete Inst., Vol. 34, March-April, 1938; Proceedings Vol. 34, p. 381.

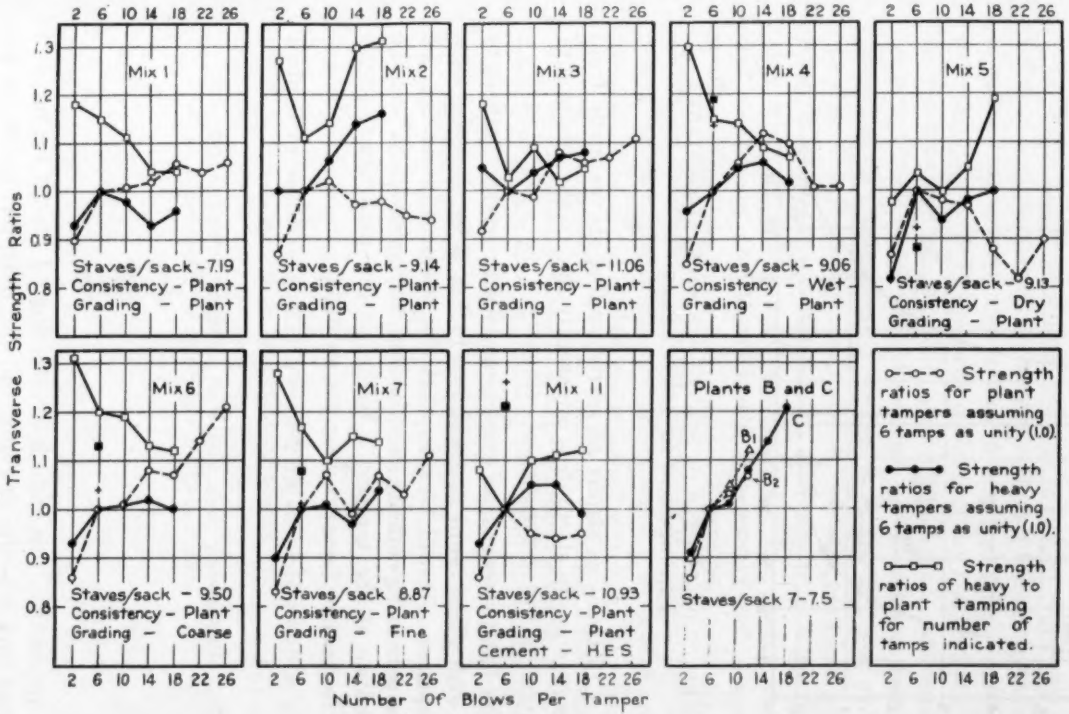

FIG. 2—RATIOS SHOWING THE EFFECT OF VARYING THE WEIGHT AND NUMBER OF BLOWS OF THE TAMPERS. THE CROSS IS THE STRENGTH RATIO FOR INDICATED MIX AT SIX PLANT TAMPS TO MIX 1, 2, OR 3 WHICHEVER HAS AN EQUIVALENT NUMBER OF STAVES PER SACK. THE SOLID SQUARE IS THE STRENGTH RATIO FOR INDICATED MIX AT SIX HEAVY TAMPS TO MIX 1, 2, OR 3 WHICHEVER HAS AN EQUIVALENT NUMBER OF STAVES PER SACK. STANDARD CEMENT USED FOR ALL MIXES EXCEPT NO. 11, FOR WHICH HIGH EARLY STRENGTH WAS USED

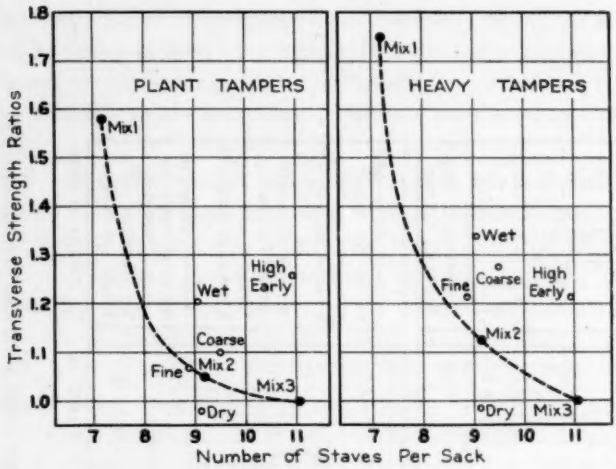

FIG. 3—EFFECT OF NUMBER OF STAVES PER SACK, CONSISTENCY AND GRADING OF AGGREGATES

tamps were 1.19 times as strong as corresponding staves made from mix 2. This increase in strength is apparently due to the addition of about one quart of water per sack of cement.

The ratios of the strengths of staves made with heavy tampers to those made with plant tampers are shown in Fig. 2 by the solid lines with open squares. It will be seen that the effect of increasing the weight of tampers is dependent upon the mix, the consistency, and the number of blows of the tampers. With but few exceptions, staves made with the heavier tampers were the stronger.

The ratios of the strengths at 2 heavy tamps to those at 6 plant tamps are as follows:

Mix Number	Strength Ratios
1	1.07
2	1.11
3	1.08
4	1.10
5	0.85
6	1.12
7	1.06
11	0.93

It will be noted that this ratio is greater than 1 except for mixes 5 and 11. Since mix 5 is the dry mix and the data for mix 11 at six tamps appear to be inconsistent, these exceptions are not thought to be important. Consequently, some economy of time with no loss in stave quality can be realized at this plant by the use of heavier tampers and fewer blows in molding.

In order to compare the effect of proportions, aggregate grading, and consistency, the strengths of all mixes at 6 tamps are expressed as a ratio of the strength of mix 3 and plotted in Fig. 3 against the number of staves per sack:

In Fig. 3 the dashed lines may be taken to represent the relations between the number of staves per sack and the strengths at six tamps of staves made from the plant grading and plant consistency. The other plotted points have been designated by the departure of the mix they represent from the plant grading and plant consistency. It will be noted from Fig. 3 that the change in consistency from dry to wet caused a very considerable change in strength. For example, using the data for the heavy tampers, the change from plant to wet consistency is apparently equivalent to a change from 9 to $7\frac{3}{4}$ staves per sack when using the plant consistency. The staves made from the wet consistency at 6 tamps had a sag of 0.011 inch as compared with 0.013 inch for the plant consistency and 0.012 inch for the dry consistency. Hence all staves were of equal straightness. The difference in added water for the normal and wet mixes was 3.5 lb. for 18 stave batches. Because these slight differences in added water can have such a marked effect on the strength, and the quantity of added water was determined only by visual inspection of staves and all staves were of about equal straightness, it cannot be said that the variations in strength apparently caused by variations in aggregate grading or type of cement are actually due to these variables. The variations in strength may just as well be due to differences in consistency. The same argument may be applied to the apparent effect of the number of staves per sack indicated by the dashed lines of Fig. 3. Hence until a method of quantitatively measuring the consistency is found, it is not possible to separate the effect of other variables, such as the grading, from the effect of the variation in quantity of water.

Discussion, to close in February, 1940* JOURNAL, *should reach A. C. I. Secretary in triplicate by Dec. 1, 1939.

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Tests of Considère Hinges Under Direct Stress, Bending, and Shear*

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SYNOPSIS

Sixteen Considère hinges were tested in 1938 as a portion of an investigation at the University of Maryland upon the principal hinge types used at present in rigid frame construction. Two concrete strengths were used with various combinations of rotation, thrust, and shear. Rotation was applied at fixed thrusts as well as simultaneously with thrust. Resistance to moment was measured for applied rotations. In addition to the two concrete strengths, three spiral percentages and shear-thrust ratios of 0, 0.2, 0.5, and 1.0 were chosen as variables.

The test results suggest a design procedure based upon, (a) preliminary proportioning of the section for axial thrust by current spiral column formulas, (b) limitation of the deformation in the extreme fibers at maximum rotation, to a permissible portion of the ultimate value, (c) design of the spiral as transverse reinforcement when the construction is subject to shear.

INTRODUCTION

Tests were begun in 1937 by the Engineering Experiment Station of the University of Maryland on reinforced concrete articulations used in arch and rigid frame construction. The results of 62 tests of Mesnager hinges without mortar covering have been published (7). Four groups were included in the 1938 program, each of which was an independent unit. The first of the 1938 group was an extension of the 1937 series to check yield point and L/r trends for the Mesnager hinge with bare bars. The second group consisted of Mesnager hinges with mortar covering tested under thrust, shear, and moment, with various

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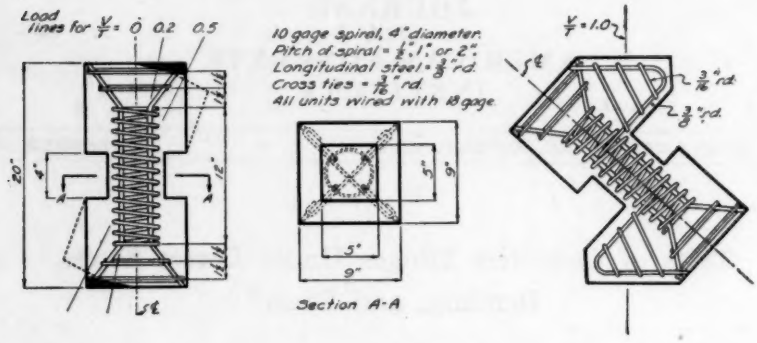


FIG. 1 (ABOVE)—DETAILS OF CONSIDERE TEST HINGES

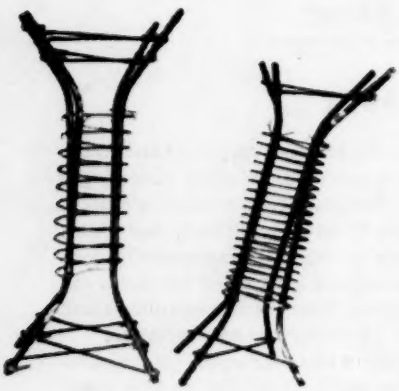


FIG. 2 (LEFT)—REINFORCEMENT UNITS PRIOR TO PLACING IN FORMS

angles of bar crossing and widths of hinge openings. The third and fourth groups were composed of Considere hinges and rollers respectively. This paper reports the results of the third group in the 1938 series of tests.

SPECIMENS

All specimens were 20 in. high and contained reinforcement units consisting of a 4-in. diameter, 10 gage spiral at ½-, 1- or 2-in. pitch and four ⅜-in. diameter longitudinal bars (Fig. 1 and 2). The dimensions of each end block were 9 x 9 x 8 in. for those tested without shear, which provided a 4-in. opening for the portion acting as a hinge. Those tested with shear were of similar dimensions but cast at angles to provide shear-thrust ratios of 0.2, 0.5, and 1.0 (Fig. 1 and 11). The hinge was 5 x 5 in. in cross-section with the exception of hinge 1 which had a 4½ in. diameter section.

The first four specimens and every three thereafter were cast from a single batch of machine mixed concrete. The concrete mixtures

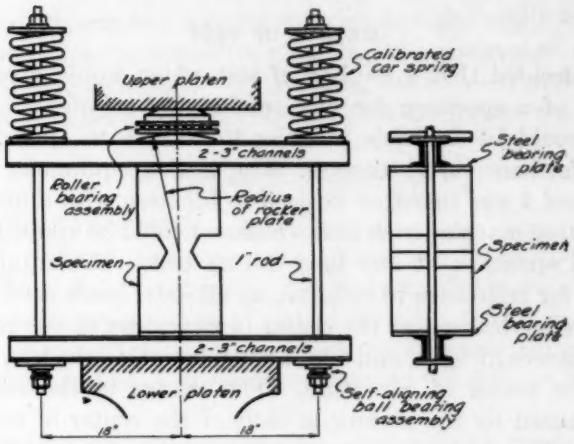


FIG. 3—METHOD OF APPLYING MOMENT TO TOP OF SPECIMEN AT ANY TIME DURING TEST

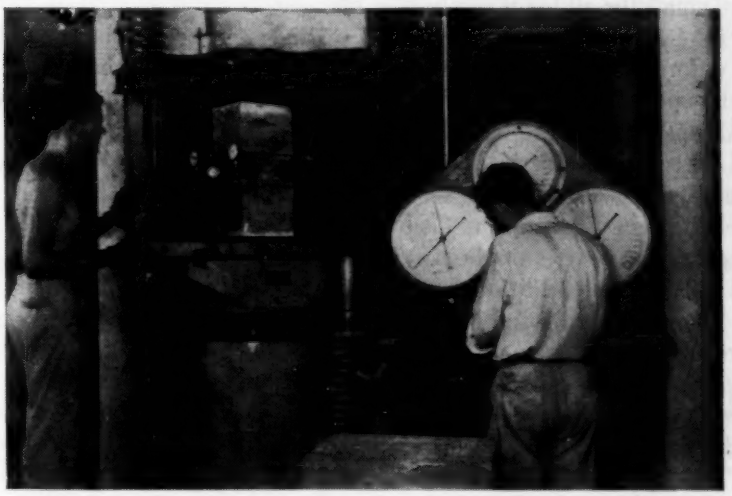


FIG. 4—MOMENT EQUIPMENT IN OPERATION

Note: The hinge is of the Mesnager type. Considere hinges were tested in the same manner.

were designed by the trial batch method to obtain a plastic mixture with a 6-to 7-in. slump. Two concrete strengths were chosen, one at about 2500 p.s.i., and the other at about 4500 p.s.i. The fine aggregate was washed sand and the coarse aggregate a washed gravel limited to that passing a ¾-in. sieve and retained on a No. 4. The cement was ordinary portland cement. All specimens were tested after 21 days cure under moist burlap.

METHOD OF TEST

It was decided that a method of test which would give the characteristics of a specimen for any combination of shear, thrust, and moment would be desirable in order to be able to approximate the actual performance in a structure. The testing equipment illustrated in Fig. 3 and 4 was therefore devised to function with a universal hydraulic testing machine such that a moment could be applied to the top block of a specimen at any load during test. The equipment was calibrated for resistance to rotation, at all axial loads used, by means of a knife edge bearing at the center of curvature of the rocker plate and at distances of ½ in. and 1 in. above, below, to the left, and to the right of the center of curvature. The change in the resistance to rotation caused by the maximum shift of the center of rotation was not measurable on the equipment. The resistance to rotation caused by application of axial loads required that corrections be made on the rotation resistance calculations for the specimens. Such corrections were small for all loads.

Fig. 4 indicates the arrangement of dials. It should be noted that the deformation at the centerline will be tensile when the neutral axis of the specimen falls between the centerline and the compressive face.

All specimens were tested with a thin cap of plaster of paris at top and bottom. A spherical bearing block placed above the roller bearing assembly was used for all tests and the roller bearing assembly was leveled at the beginning of each test.

Specimens 1 to 4 inclusive were tested under direct thrust only and without the additional equipment. For all other specimens, the top block was subjected to rotation at various applied loads. It must be recognized that the ultimate loads are not truly comparable excepting for those cases in which identical testing procedure was used and in which no major damage was done by rotation prior to the ultimate load. Specimens 1 to 5, and 12 to 14 may be so considered.

The testing procedure for specimen 5 was intended to provide a loading condition similiar to that occuring upon removal of forms or arch centering, the rotation and thrust being applied simultaneously. The rate of thrust application was set at about 500 lb. per second. The rotation was applied as rapidly as possible by means of the additional equipment. After the final dial and spring deformation readings, the moment and thrust were simultaneously released to the former value of thrust. The entire test procedure may be determined from Fig. 6.

All remaining specimens were tested by applying and releasing rotation at fixed thrust. The detailed procedure may be determined from a study of Fig. 6, 7, 8 and 9.

At no time during test was the rotation brought arbitrarily back to zero. In other words, any set in rotation upon removal of moment or load was allowed to remain. All moment was released prior to testing the specimens to the ultimate, although a set in rotation was invariably present.

DISCUSSION OF RESULTS

Effect of Dial Arrangement

All deformations and rotations across the hinge opening were obtained from dials arranged in the manner shown in Fig. 4. For such an arrangement of dials it should be noted that:

(a) Rotations of the top block will be recorded correctly unless failure of an end block disturbs the position of the dials or their contact plane,

(b) Deformations recorded by the dials will be produced over a length equal to the 4 in. length within the opening plus an unknown portion acting within each end block. Since the dials were attached within an inch of the opening, any deformation of the end blocks as a unit would be negligible,

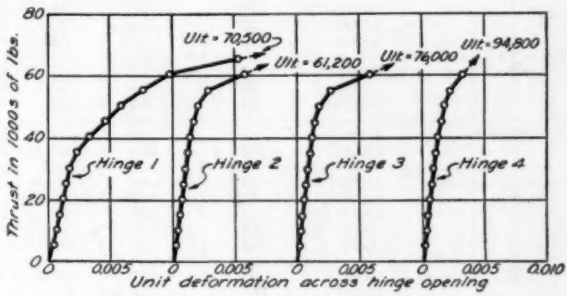


FIG. 5—CONSIDERE HINGES UNDER DIRECT THRUST $\left(\frac{V}{T} = 0\right)$.

Concrete strength = 2500 p.s.i. Spiral steel percentage = 1.43, 0.72, 1.43, 2.86 for hinges 1, 2, 3 4 respectively. 4½ in. diam. section for hinge 1, all others are 5 in. by 5 in. Longitudinal steel = 4⅜ in. rd.

(c) For direct stress only, the portion deformed within each end block will increase with the magnitude of the applied load. Under combined direct stress and rotation, the portion acting within the end block will increase on the high compression side and decrease on the low compression side (until tension exists) for increasing rotation.

Effect of Spiral Percentage

Deformation curves for No. 1 to 4 (Fig. 5) show the effect of variation in spiral percentage for direct thrust only. The action is typical of spirally reinforced concrete columns subjected to axial load. Speci-

mens 5 to 7 and 11 to 13 showed no appreciable effect, due to variation in spiral percentage, in specimens during rotation (Table 1 and Fig. 6 and 8). The ultimates for specimens 12 and 13 may be compared since the method of test was the same and no appreciable damage occurred during rotation.

TABLE 1—SUMMARY OF TEST RESULTS FOR REINFORCED CONCRETE HINGES OF THE CONSIDERE TYPE

Specimen No.	Spiral Steel % of Core Vol.	Cylinder Strength at Test of Hinge (p.s.i.)	Ratio of Shear to Thrust (V/T)	Moment and Rotation Data				Ultimate Load (Moment Released Prior to Loading to Ult.) (Pounds)
				Load at Which Rotation Was Applied (Pounds)	Range of Rotation (Radians)	Change in Rotation (radians)	Maximum Moment Developed (in. lb.)	
a	b[1]	c[2]	d	e[3]	f	g	h[4]	i[5]
1	1.43	2500	0	No rotation applied to Nos. 1 to 4				70500
2	0.72	↓	↓					61200
3	1.43							76000
4	2.86							94800
5	0.72	2680	0	5000 to 8900	−0.0008 to +0.0061	+0.0069	+25000	70800
				10000 to 14200	2 87	85	27000	
				15000 19650	13 94	81	26000	
6	2.86		0	20000	− 2 72	74	35000	71000
				40000	22 88	66	39500	
				60000	26 88	62	19500	
7	1.43		0	20000	− 2 71	73	36500	68200
				40000	20 115	95	35500	
				60000	34 64	30	12500	
8	1.43	2810	0.2	15000	− 9 72	81	17500	40500
				15000	− 34 − 122	− 88	− 16500	
9	1.43		0.5	10000	24 80	56	9500	33200
				10000	62 − 10	− 72	− 14500	
				10000	45 120	75	12000	
				20000	98 172	74	4000	
				20000	143 80	− 63	− 5500	
10	1.43		1.0	10000	50 137	87	5500	18700
				10000	118 56	− 62	− 7000	
11	1.43	4490	0	20000	5 87	82	32500	73700
				40000	34 110	76	35000	
				40000	72 9	− 63	− 30000	
				60000	43 102	59	18000	
12	0.72		0	25000	4 106	102	33500	86400
13	2.86		0	25000	0 112	112	46500	99200
14	1.43	4380	0.2	20000	− 12 60	72	34000	69400
				20000	9 − 63	− 72	− 25500	
15	1.43		0.5	15000	5 − 69	− 74	− 36000	44200
				15000	− 8 65	73	17500	
16	1.43		1.0	10000	26 − 44	− 70	− 24500	23200
				10000	13 86	73	11500	

[1]Longitudinal bars = 4⅜″ rd. pl. (av. Y. P. = 48,350 p.s.i.); spiral = 10 gage, 4″ diam., @ ½″, 1″, or 2″ pitch. 4″ hinge opening with 5″ by 5″ cross-section (4½″ diam. for No. 1 only).

[2]Average of 4—4½ by 9 cyl. for No. 1 to 4, av. of 3 for all others.

[3]Rotations were applied at constant load excepting for No. 5 for which thrust and moment were applied simultaneously.

[4]Positive moment is counterclockwise with shear acting to the right (or clockwise with shear acting to the left.)

[5]Spirals were broken in No. 1 to 4 only. No. 7 & 11 developed end block failures prior to that of the concrete in the opening.

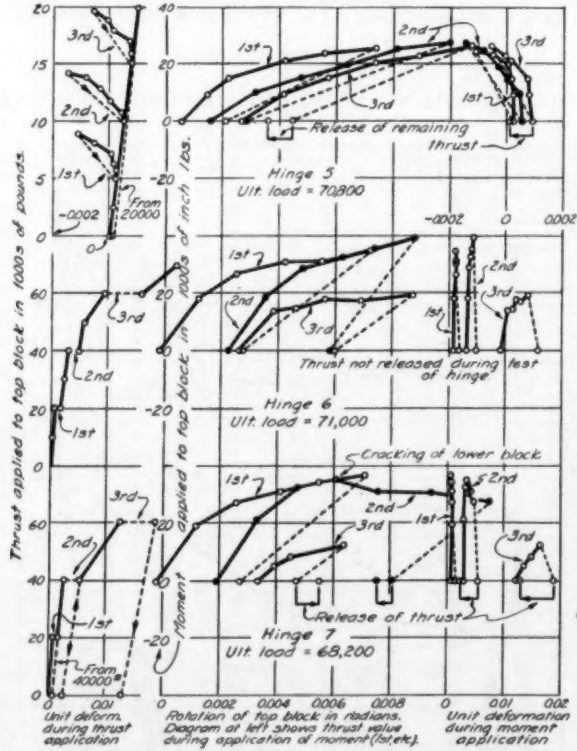

Thrust applied to top block in 1000's of pounds
Moment applied to top block in 1000's of inch lbs.
Hinge 5
Ult. load = 70,800
Release of remaining thrust
Hinge 6
Ult. load = 71,000
Thrust not released during test of hinge
Hinge 7
Ult. load = 68,200
Cracking of lower block
Release of thrust
From 20000
From 40000
Unit deform. during thrust application
Rotation of top block in radians. Diagram at left shows thrust value during application of moment (1st, etc.).
Unit deformation during moment application

FIG. 6—CONSIDERE HINGES UNDER THRUST AND MOMENT $\left(\frac{V}{T} = 0\right)$

Concrete strength = 2680 p.s.i. Spiral steel percentage for 5, 6 and 7 = 0.72, 2.86 and 1.43 respectively.

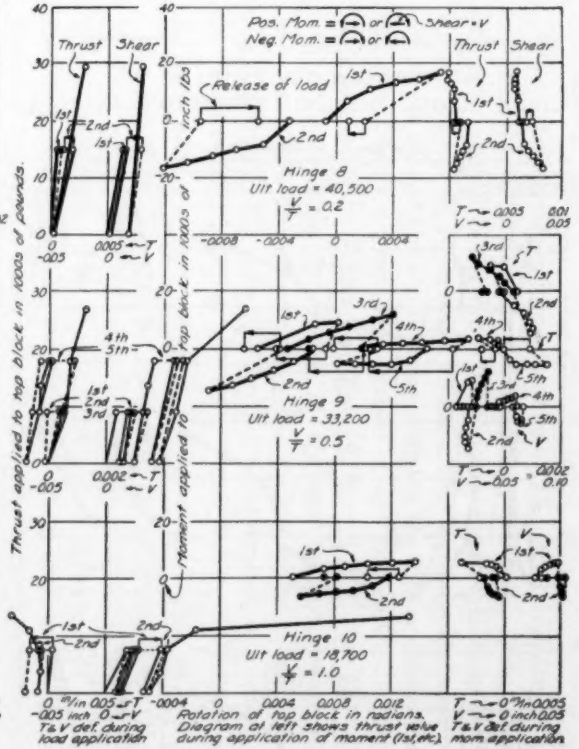

Thrust applied to top block in 1000's of pounds
Moment applied to top block in 1000's of inch lbs.
Pos. Mom. = or Shear +V
Neg. Mom. = or
Thrust
Shear
Release of load
Hinge 8
Ult load = 40,500
V/T = 0.2
Hinge 9
Ult load = 33,200
V/T = 0.5
Hinge 10
Ult load = 18,700
V/T = 1.0
T&V def. during load application
Rotation of top block in radians. Diagram at left shows thrust value during application of moment (1st, etc.).
T&V def. during mom. application

FIG. 7—CONSIDERE HINGES UNDER THRUST, SHEAR, AND MOMENT

Concrete strength = 2810 p.s.i. Spiral steel percentage = 1.43.

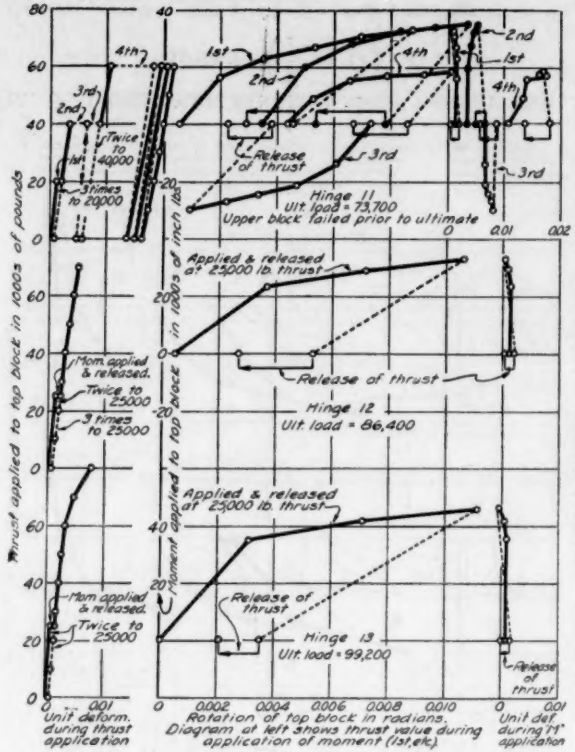

Thrust applied to top block in 1000's of pounds
Moment applied to top block in 1000's of inch lbs.
Hinge 11
Ult. load = 73,700
Upper block failed prior to ultimate
Hinge 12
Ult. load = 86,400
Hinge 13
Ult. load = 99,200
Applied & released at 25,000 lb. thrust
Release of thrust
Mom. applied & released.
Twice to 25000
3 times to 25000
Twice to 40000
3 times to 20000
Unit deform. during thrust application
Rotation of top block in radians. Diagram at left shows thrust value during application of moment (1st, etc.)
Unit def. during "M" application

FIG. 8—CONSIDERE HINGES UNDER THRUST AND MOMENT $\left(\frac{V}{T} = 0\right)$

Concrete strength = 4490 p.s.i. Spiral steel percentage = 1.43, 0.72 and 2.86 for hinges 11, 12 and 13 respectively.

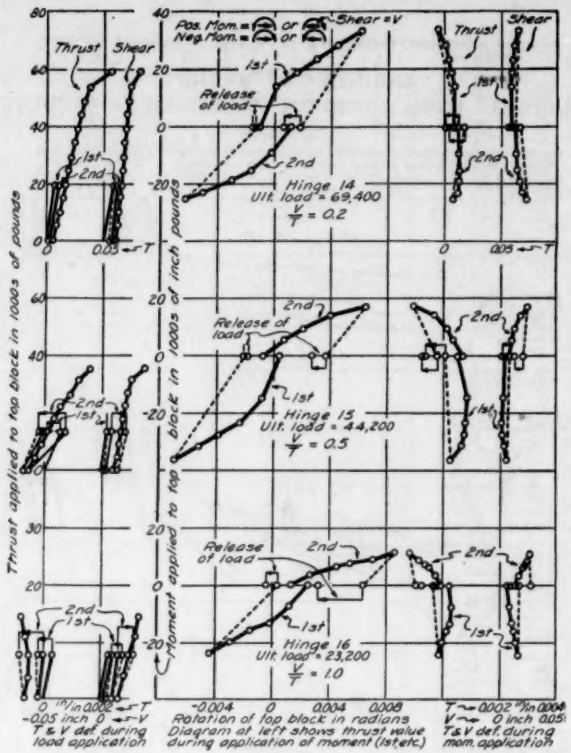

Thrust applied to top block in 1000's of pounds
Moment applied to top block in 1000's of inch pounds
Thrust
Shear
Pos. Mom. = or
Neg. Mom. = or
Hinge 14
Ult. load = 69,400
V/T = 0.2
Hinge 15
Ult. load = 44,200
V/T = 0.5
Hinge 16
Ult. load = 23,200
V/T = 1.0
Release of load
T & V def. during load application
Rotation of top block in radians
Diagram at left shows thrust value during application of moment (1st, etc.)
T & V def. during mom. application

FIG. 9—CONSIDERE HINGES UNDER THRUST, SHEAR AND MOMENT

Concrete strength = 4380 p.s.i. Spiral steel percentage = 1.43.

Concrete Strength

Since the Considere hinge functions as a spirally reinforced concrete column, the effect of different grades of concrete upon the ultimate strength should be obvious when such construction is used under axial load only. Even after rotation has been applied and released, a distinct gain at the ultimate is shown for the higher strength concrete in those cases for which the method of test is such that a comparison may be made (No. 5 and 12, 8 and 14, 10 and 16, Table 1).

During rotation, the action of the two concretes was essentially the same and the difference in the resistance to rotation was very slight (Table 1, Fig. 6 to 9). Further consideration of rotation is made in the section on rotation.

Effect of Shear

Deformation diagrams for specimens tested under shear (Fig. 7 and 9) show large permanent deformations, particularly in shear, at relatively low loads. For shear-thrust ratios in excess of 0.2 it seems essential to increase the percentage of spiral steel in order to prevent diagonal tension failures. The large permanent tensile deformations for specimens 9, 10, 15, and 16 (Fig. 7 and 9) during rotation are due to the passing of the yield point of the tensile steel since the thrust was not sufficient for V/T ratios of 0.5 and 1.0 to keep the tensile steel ($p = 0.0088$) within the yield point at the larger rotations.

The visible manner of failure for specimens 8 and 14 ($V/T = 0.2$) was similiar to that for specimens under direct thrust only, with spalling of the shell and crushing of the core. For all other shear specimens the visible manner of failure was that of diagonal tension (Fig. 11). If the spiral is considered as transverse reinforcement and the useful limit stress taken as 60,000 p.s.i., the maximum resistance to diagonal tension failure provided by the spiral is 340 p.s.i. for the 1-in. pitch. Table 2 provides calculated maximum tensile stresses in the concrete at the ultimate load for all specimens tested with shear. It appears that design against diagonal tension failure might be made in the usual manner for members subjected to direct stress and shear, utilizing the spiral as transverse reinforcement.

Rotation

The results plotted in Fig. 6 to 9 indicate that (a) a linear relationship between moment and rotation angle does not exist up to maximum rotations likely to be encountered in such construction, (b) the resistance offered to a given rotation varies but slightly with increases in thrust up to 40,000 lb., (c) permanent axial deformations produced at 20,000 lb. thrust (25,000 for No. 12 and 13) with highest rotation

FIG. 10 (LEFT)—TYPICAL MANNER OF FAILURE FOR HINGES TESTED UNDER DIRECT THRUST AND FOR THE SHEAR THRUST RATIO OF 0.2

NOTE: No. 7 is Considere hinge No. 11.

FIG. 11 (BELOW)—TYPICAL DIAGONAL TENSION FAILURES FOR HINGES TESTED UNDER SHEAR THRUST RATIOS OF 0.5 AND 1.0

NOTE: No. 11 is Considere hinge No. 15, No. 12 is No. 16.

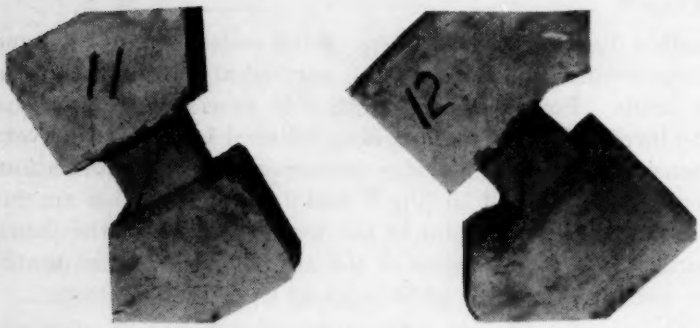


TABLE 2—CALCULATED TENSILE STRESS IN CONCRETE AT MID-SECTION FOR SPECIMENS TESTED WITH SHEAR

Specimen No.	Cylinder Strength at Test of Hinge (p.s.i.)	Ultimate Load		Max. Shearing Unit Stress (p.s.i.)	Compr. Unit Stress (p.s.i.)	Max. Textile Unit Stress		Tensile Capacity of Spiral (p.s.i.)	Tensile Stress in Concrete at Ult. (p.s.i.)	Ratio of Tensile Stress to Cylinder Strength
		V (Lbs.)	T (Lbs.)			Stress (p.s.i.)	Angle with centerline (Degrees)			
a[1]	b	c	d	e[2]	f[2]	g	h	i[4]	j	k
8	2810	7940	39700	475	1360	150	72° 30′	340	—	—
9	↓	14850	29700	890	1020	520	60° 00′	↓	180	0.064
10	↓	13200	13200	605	450	600	60° 10′	↓	260	0.093
14	4380	13600	68000	815	2460	250	73° 15′	↓	—	—
15	↓	19800	39600	1190	1440	670	60° 30′	↓	330	0.075
16	↓	16400	16400	750	600	730	60° 30′	↓	390	0.089

[1]Visible type of failure of Nos. 9, 10, 15 & 16 was diagonal tension. Nos. 8 & 14 appeared to fail in compression on core area.

[2]Computed as 3/2 the average. This may be somewhat high for Nos. 10 & 16 which showed tensile deformation at the ℄ as long as dials were attached.

[3]"n" taken as 30,000 divided by cylinder strength.

[4]Useful limit stress taken as 60,000 p.s.i.

are not disconcerting, while those produced at 40,000 or 60,000 lb. are definitely large.

Furthermore, crushing of the outside fibers on the high compression side became visible at the highest rotation for the 40,000 lb. thrust and spalling was evident at 60,000 lb. when rotation was applied. Specimen 11 (cylinder strength = 4490) shows evidence that the initial crushing did not extend to any great depth, since the two applications and release of the 40,000 lb. thrust, subsequent to rotation, showed no permanent deformation. After rotation at 60,000 lb., however, each application and release of thrust shows a large permanent set. Specimen 7 (cylinder strength = 2680) had the same deformation at 40,000 lb. after a single release and reapplication of thrust subsequent to rotation.

Axial design loads based on the A. C. I. code would be 24,000 lb. for the low strength concrete and 34,000 lb. for the high strength when based on 20,000 p.s.i. in the steel (average yield point of steel = 48,350 p.s.i.). Although rotation data at the design loads would have been desirable it appears safe to conclude, for these hinges, that rotations up to 0.01 radian super-imposed upon the design load would not have produced distress over a critical portion of the cross-section for either concrete.

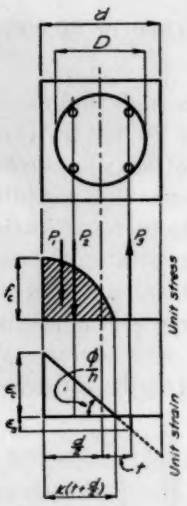
DESIGN

Construction of this type is subjected to an almost irresistable rotation which is fixed in value. After this rotation has taken place, it does not *follow up* on the concrete when beyond the elastic range of deformation. It might be permissible in such a case to allow larger deformations at the outer fibers than would occur under elastic conditions.

The following method of design[1] is suggested as a possibility:

1. Make a preliminary design of the section for axial thrust by current spiral column design formulas,
2. Limit the deformation in the extreme fibers at maximum rotation to a permissible portion of the ultimate value determined from flexural tests.
3. Check the diagonal tension stress as for a member under combined shear and direct stress, with the spiral acting as transverse reinforcement.

[1]Considerable material is available on reinforced concrete subjected to direct stress and flexure (1,2, 6 and 8) with respect to ultimate loads and design. It should be noted that the tests herein reported differ from previous ones in the following respects (a) the member was extremely short, (b) the application and release of rotation was made at constant loads below the ultimate, and (c) the application of load to failure was made without applied moment. The problem of design is that of limiting deformation in place of stress.



Total load = $P = P_1 + P_2 - P_3$

$$= q f'_c b k(t+\tfrac{t}{2}) + f'_s a'_s - f''_s a''_s \qquad (1)$$

in which, $q f'_c$ = av. concrete stress.
b = breadth of section.
d = depth of section.
$k(t+\frac{t}{2})$ = dist. to N.A. from compr. face

Also $\dfrac{\phi}{h} = \dfrac{\epsilon_c}{k(t+\frac{t}{2})}$ in which, h = hinge opening. (2)

From 1 & 2, the following formulas may be developed if it is assumed that, (a) distribution of stress in the concrete conforms to a second degree parabola, (b) reinforcement is symmetrical.

Max deformation limited to the ultimate value, $\epsilon_c = \epsilon'_c$; $f'_c = f'_c$

$$\frac{P}{A_g} = \frac{2}{3} f'_c k(\frac{t}{d} + \frac{t}{2}) + \frac{p_g}{2}\left[f_y - (\frac{1-k}{k})\epsilon'_c E_s\right] \qquad (3)$$

$$\phi(\frac{d}{h}) = \frac{\epsilon'_c}{k(\frac{t}{d}+\frac{t}{2})} \qquad (4)$$

Max deformation limited to 50% of the ultimate value, $\epsilon_c = 0.5\epsilon'_c$; $f'_c = 0.75 f'_c$

$$\frac{P}{A_g} = \frac{5}{12} f'_c k(\frac{t}{d} + \frac{t}{2}) + \frac{p_g \epsilon'_c E_s}{4k}\left[2k + \frac{(\frac{t}{d} - \frac{t}{2})}{(\frac{t}{d}+\frac{t}{2})} - 1\right] \qquad (5)$$

$$\phi(\frac{d}{h}) = \frac{\epsilon'_c}{2k(\frac{t}{d}+\frac{t}{2})} \qquad (6)$$

For the reinforcement shown, $\frac{t}{d}$ may be taken as $0.32\frac{D}{d}$ without appreciable error. Also, $\frac{D}{d} = 0.8$ for most construction

FIG. 12—FORMULAS USED IN CONSTRUCTION OF DESIGN CHARTS

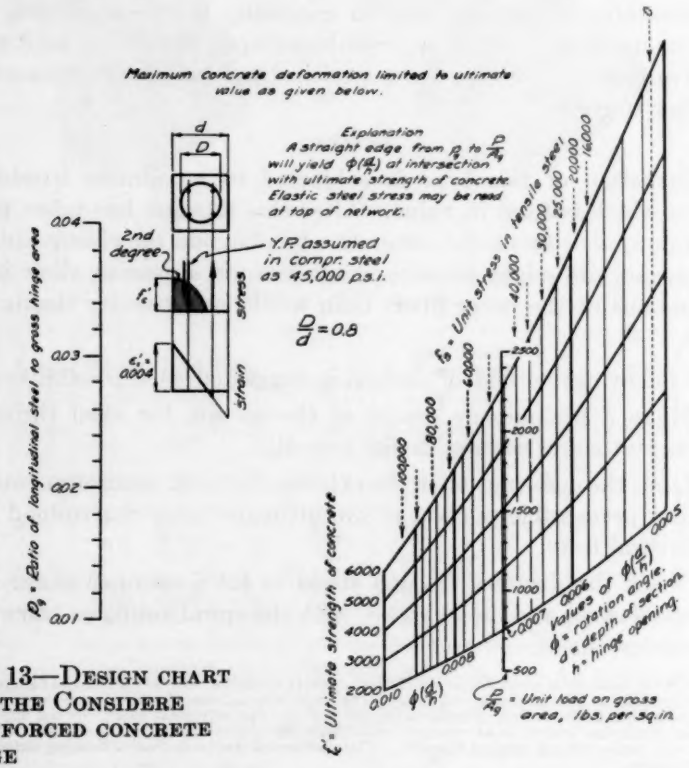


FIG. 13—DESIGN CHART FOR THE CONSIDERE REINFORCED CONCRETE HINGE

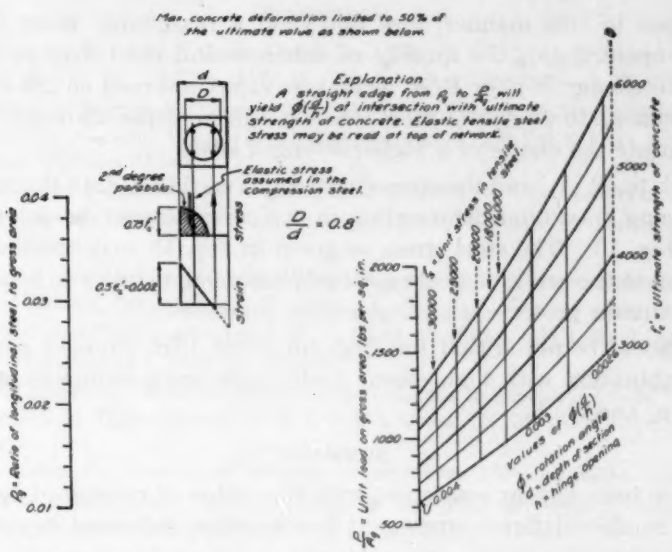


FIG. 14—DESIGN CHART FOR THE CONSIDERE REINFORCED CONCRETE HINGE

Values for the ultimate unit deformation of concrete have been discussed by other investigators with respect to reinforced concrete subjected to flexure or direct stress and flexure (1, 2). Such discussion has suggested values varying from 0.0015 to 0.0070 depending upon the mixture, type of loading, and degree of confinement of material.

For short hinge openings the material in a Considere hinge will be more confined than for a cylinder or beam. It appears proper to recommend values for such a condition only after a thorough study of existing literature with subsequent tests if necessary.

The formulas in Fig. 12 are developed on a basis of a parabolic[2] (second degree) distribution of stress with a linear distribution of strain at the ultimate deformation or at 50 per cent of the ultimate deformation, irrespective of actual values. The charts (Fig. 13 and 14), however, are for maximum rotations at an assumed ultimate unit deformation of 0.004 or design rotations restricted to deformations of 50 per cent of 0.004.

For design, p_g and P/A_g may be determined first in the customary manner for reinforced concrete spiral columns from an assumed concrete strength and grade of steel. With these values, $[\phi(d/h)]$ is obtained from Fig. 14 corresponding to the ultimate strength of the concrete used in determining P/A_g. If the value of $[\phi(d/h)]$

[2]Other distributions of stress may be adapted if desired. (See reference 2 and discussions). The choice of the second degree parabola is made solely for purpose of demonstration.

obtained in this manner does not yield a reasonable value for the hinge opening (h), the quality of concrete and steel may be varied with no change in p_g or P/A_g as long as values are read on the straight line from p_g to P/A_g. A steel stress in excess of the allowable would necessitate the choice of a higher strength steel.

With p_g, P/A_g, and the concrete strength used in Fig. 14 the rotation producing maximum deformation in the concrete may be determined from Fig. 13. The steel stress as given in Fig. 13 may necessitate a revision in the grade of steel previously assumed, in order to be assured of maximum protection against excess rotations.

It should be noted that for large rotations, high strength concretes in combination with high elastic limit steels are a requirement for a given p_g and P/A_g.

SUMMARY

These tests appear consistent with the action of reinforced concrete under combined direct stress and flexure when deformed beyond the elastic range.

For design it is suggested that the section be proportioned to carry the axial load in accordance with current spiral column formulas with a limitation on the compressive deformation in the extreme fiber at maximum rotation, based on a permissible portion of the ultimate value. It is further suggested that the diagonal tension stress be checked as in a member subjected to shear and direct stress, with the spiral acting as transverse reinforcement.

ACKNOWLEDGMENTS

This research is part of a continuing cooperative project between the Maryland State Roads Commission and the University of Maryland Engineering Experiment Station, of which Dean S. S. Steinberg is Director.

Appreciation is expressed to Prof. H. J. Gilkey, Iowa State College; Prof. F. E. Richart, University of Illinois; Rear Admiral Moreell, C. E. C., U. S. Navy; and Albin Gemeny, Bureau of Public Roads for comments or suggestions relative to the testing equipment and method of test.

Cement, sand, and gravel were furnished by the Maryland State Roads Commission. Billet steel bars and spiral steel were furnished by the Laclede Steel Co., St. Louis, Mo.

Credit is due George Crum and William B. Davis, Jr., laboratory assistants, for unusual care taken in the construction of the specimens and aid given during the tests.

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High Yield-Point Steel as Tension Reinforcement in Beams*

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SYNOPSIS

Results of tests of 32 rectangular concrete beams reinforced with four different types of high yield-point steels are presented in this report. The beams had an effective depth of 12 in., a width of 12 in. and a distance center-to-center of supports of 9 ft. The four types of steel used were: (1) hard grade steel, (2) nickel steel (one beam only) (3) square twisted bars, and (4) "twin-twisted and stretched" bars.

Results show that when a concrete beam is reinforced against diagonal tension failure the strength is determined by the total yield strength of the steel (steel area times yield-point stress) and not by the type of steel.

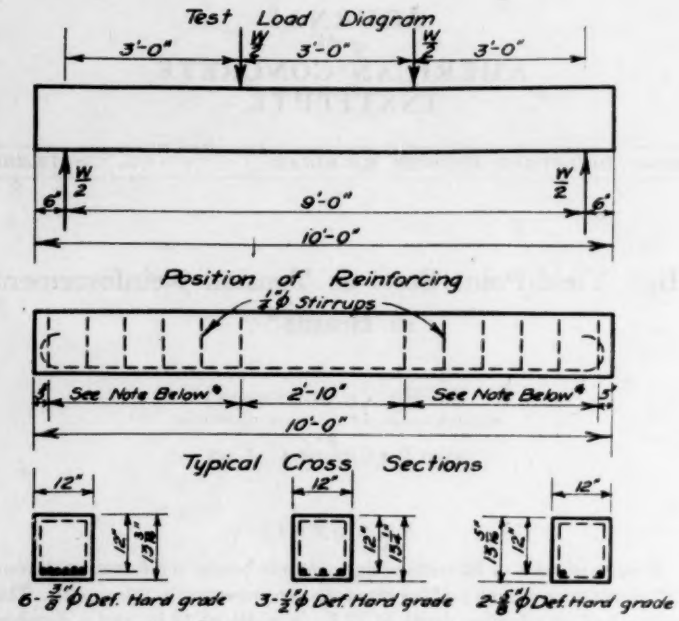
FOREWORD AND ACKNOWLEDGMENT

This investigation, sponsored by the Concrete Reinforcing Steel Institute, was started in 1937 under the direction of Inge Lyse, formerly Research Professor of Engineering Materials at Lehigh University and now Professor of Reinforced Concrete and Solid Bridges at the Norges Tekniske Hoiskole at Trondheim, Norway. The investigation was a regular research project of the Fritz Engineering Laboratory, which is under the administrative direction of Prof. Hale Sutherland, Head of the Department of Civil Engineering. Howard Godfrey, Engineer of Tests at the Fritz Engineering Laboratory, contributed continuous assistance and advice during the entire investigation.

*Received by the Institute, March 27, 1939.

†Assistant Professor of Civil Engineering, and Assistant Director in charge of research at the Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pa.

‡Concrete Reinforcing Steel Institute Research Fellow in immediate charge of investigation.



STIRRUPS

Beam Number	Number of Stirrups	Beam Number	Number* of Stirrups	Beam Number	Number* of Stirrups
A-1	6	C-3	14	N-1	14
A-2	6	C-4	15	I-1	6
A-3	6	T-1	6	IS-1	6
B-1	6	T-2	6	IS-2	11
B-2	6	T-3	6	IS-3	14
B-3	6	ST-1	11	IS-4	15
C-1	6	ST-2	14		
C-2	11	ST-3	14		

*NOTE: Number of ¼ϕ def. stirrups evenly spaced at each end of the beam.

FIG. 1—LOAD DIAGRAM AND LOCATION OF STEEL

INTRODUCTION

Purpose—The purpose of this investigation is to study the behavior of various types of high yield-point steels as tension reinforcing in concrete beams.

The question of adopting increased allowable unit stresses for high yield-point strength steel reinforcing is of current interest among designing engineers. In some localities higher stresses have been

allowed for special types of steels in which the yield point has been raised by simultaneously stretching and twisting two round bars together.

A previous investigation considered principally "twin-twisted and stretched" bars in comparison with structural grade carbon steel [1,2]. The present program has been designed to coincide with the previous tests in regard to dimensions of specimens and strength of concrete so that the data from both sources would be directly comparable.

TEST PROGRAM

Thirty-two beams were made for this program. Several sizes of each type of bar were used except in the case of the nickel steel. The variables include the type of steel, the percentage of steel, and the size of steel. Table 1 shows the type, size, and amount of reinforcing used in each beam. Modulus tests on concrete and steel were also determined. The general dimensions of the test beams and loading arrangement are shown in Fig. 1. A photograph of a typical beam in the testing machine prior to loading is shown in Fig. 2.

Steel—Physical properties of the steels used are given in Table 2. The various types of bars are shown in Fig. 3. The first bar on the left is the ⅝-in. nickel steel bar, the next four bars from the left are the hard grade bars, the next three are the square twisted bars, and the last five are the various sizes of "cold-twisted and stretched" bars The hard grade deformed bars were furnished by the Truscon Steel Co. Youngstown, Ohio. The yield point was noted by the "drop of the beam" method. Nickel steel for one beam was furnished by the International Nickel Co. of Bayonne, New Jersey. The yield point was determined by the A. S. T. M. offset method of 0.2 per cent elongation on both the nickel and square twisted steel. The square twisted steel was donated by the Bethlehem Steel Co., of Bethlehem, Pa. The No. 1 and No. 2 Isteg bars "twin-twisted and stretched" were purchased.

Other ⅜-in.$\phi\phi$, ½-in.$\phi\phi$, and ⅝-in.$\phi\phi$ "twin-twisted and stretched" bars were donated by the Bethlehem Steel Co. and were manufactured by methods identical with the bars used in the previous investigation[1]. Coupons from all "twin-twisted and stretched" bars were cut in 2½ ft. lengths and welded for 2-in. on each end to enable the bars to work together. The yield point was obtained by the A. S. T. M. offset method.

[1]Isteg Steel for Concrete Reinforcement" by D. B. Steinman, JOURNAL, Amer. Concrete Inst., Nov. 1935; Proceedings Vol. 32, p. 183.

[2]"The Modular Ratio—A New Method of Design Omitting "*m*", *Concrete and Constructional Engineering*. Mar. 1937. p. 189. K. Hajnal-Konyi.

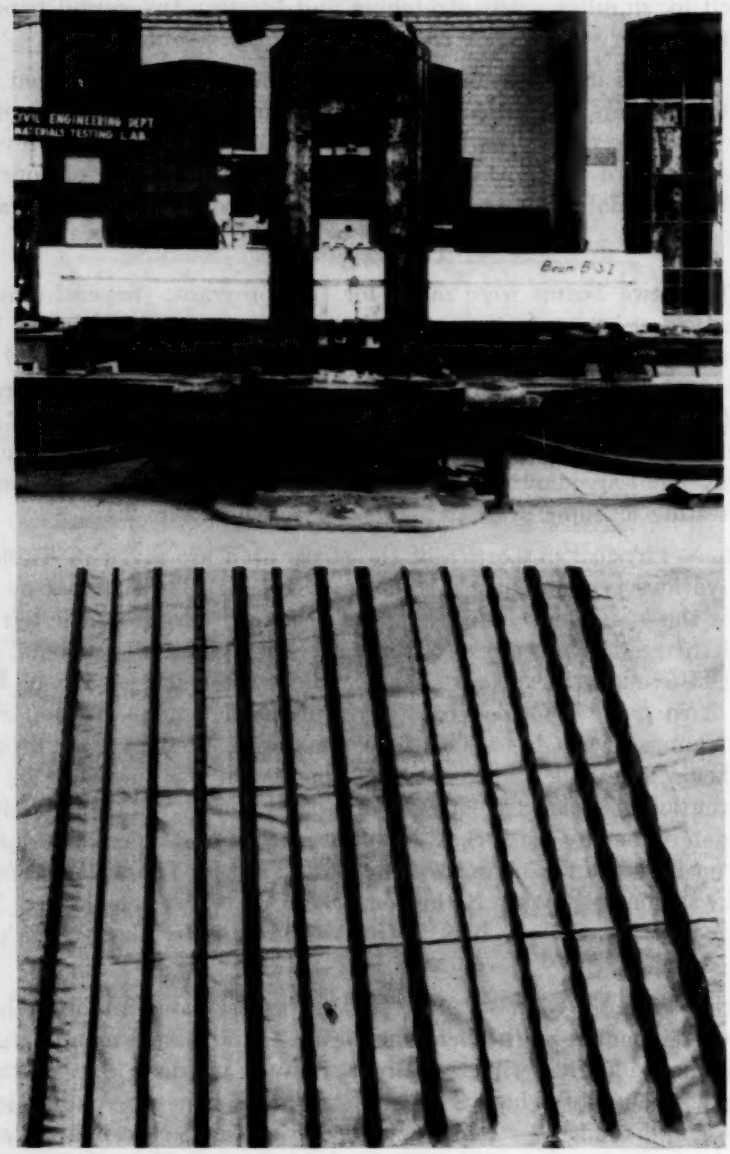


FIG. 2 (TOP)—TYPICAL BEAM IN TESTING MACHINE

FIG. 3—TYPES OF BARS TESTED

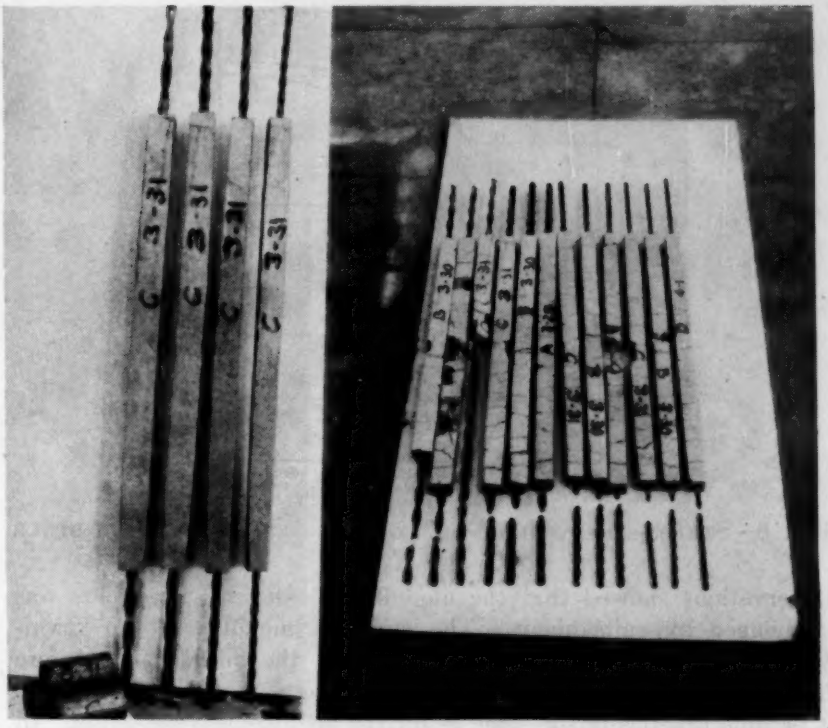


FIG. 4 (LEFT)—FOUR SPECIMENS BEFORE TESTING

FIG. 5 (RIGHT)—BARS AFTER REMOVAL FROM TESTING MACHINE

A series of tests was made to determine the effect of embedment on square twisted and "twin-twisted and stretched" bars. Three test specimens were made for each of the following bar sizes: ½-in. square twisted, ⅝-in. square twisted, ¾-in. square twisted, and ½-in$\phi\phi$ "cold twisted and stretched." These bars were embedded in the center of a square concrete block 42 in. long with a ¾-in. coverage at the nearest face. Fig. 4 shows four of the specimens before testing. The "twin-twisted and stretched" bars were welded for a few inches on each end to keep them working uniformly in the grips of the machine The deflection in the 40-in. gage length was measured by two Ames dials reading to the nearest 1/1000 in.

Fig. 5 shows the bars just after removal from the testing machine. The concrete spalled off to a greater degree in the "twin-twisted and stretched" bars than in the square twisted bars, because longitudinal cracks developed in these bars in addition to the transverse cracks.

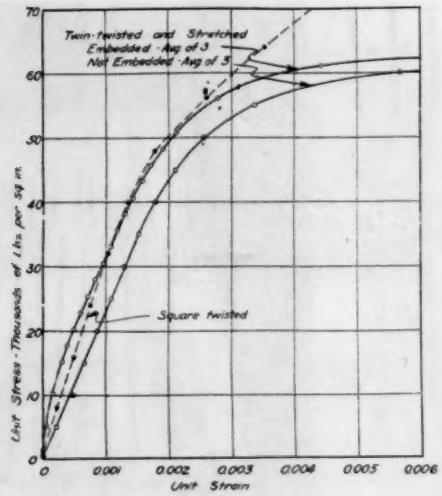


FIG. 6—STRESS-STRAIN DIAGRAMS FOR TWISTED REINFORCING STEEL

Observations showed that the modulus of square twisted bars was unchanged by embedment. The apparent modulus of the "twin-twisted and stretched" bars was raised in the initial range before failure of concrete in tension but at stresses greater than 15,000 p.s.i. it became practically the same as for the unembedded condition. In the working stress range the modulus of the "twin - twisted and stretched" bars was approximately 22,000,000 p.s.i. in both the unembedded and embedded tests, as can be determined by Fig. 6.

Concrete—The concrete was designed for 3300 p.s.i. at 28 days to correspond with the previous investigation[1].

A cement-water ratio by weight of 1.28 was used with 300 lb. of water per cubic yd. of concrete to give the desired workability. Pit sand from northern New Jersey was used for fine aggregate. The ⅜-in. and ¾-in. crushed limestone rock used as coarse aggregate was donated by the Bethlehem Steel Corp., Bethlemem, Pa. The cement was donated by the Lehigh Portland Cement Co. The proportion of sand to coarse aggregate was established at 1:2 and the proportion of ⅜-in. coarse to ¾-in. coarse was made 1:2 also.

Ten control cylinders were made for each pair of beams for the first 20 beams. For each of the last 12 beams five control cylinders were made and no strain readings were recorded. The average 28-day compressive strength of the cylinders for the first 20 beams is 3190 p.s.i., for the last 12 beams 3220 p.s.i.

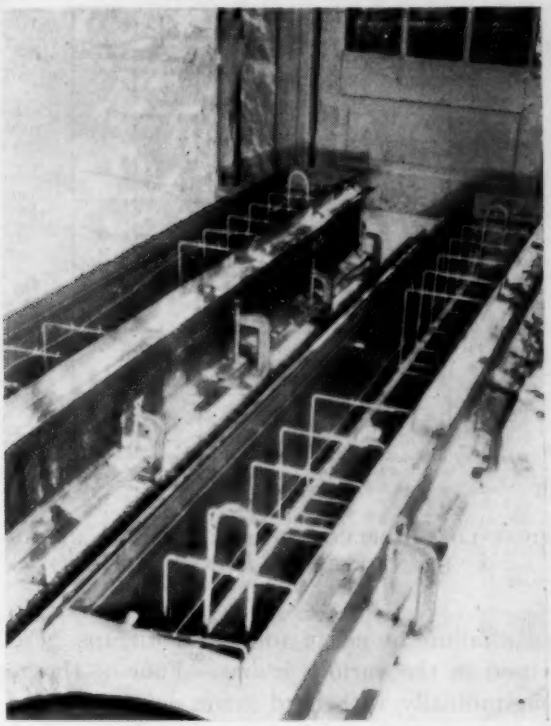


FIG. 7—REINFORCING STEEL IN FORMS

Beams—The beams had an effective depth of 12-in., a width of 12 in., and an overall length of 10 ft. Supports were nine feet center-to-center and third-point loading was used as shown in Fig. 1. The center of gravity of the steel was adjusted to exactly 12 in. by using various screeds which would give this desired depth. The steel was wired together before it was placed in the steel forms which are shown in Fig. 7.

In the first 20 beams an 8-in. stirrup spacing was used. Twelve of these beams were reinforced with hard grade deformed bars, 6 with square twisted bars, and 2 with "twin-twisted and stretched" bars. For beams with nearly equal steel areas diagonal tension failure resulted in 5 out of 6 beams with square twisted bars, both of the beams with "twin-twisted and stretched" bars, but in only 3 out of 6 beams with straight bars. This indicated a slight tendency toward diagonal failure in the case of beams with the square twisted bars and "twin-twisted and stretched" bars. The last 12 beams were designed to eliminate

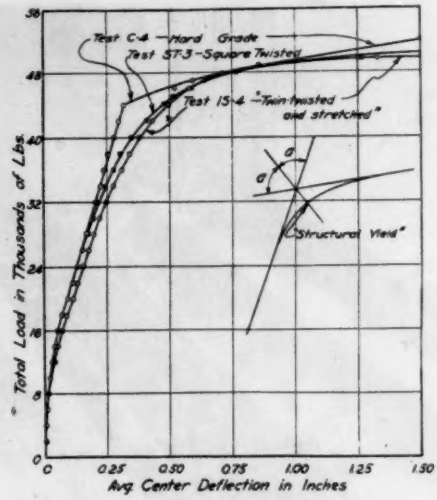


FIG. 8—TYPICAL LOAD DEFLECTION DIAGRAMS FOR BEAMS WITH DIFFERENT TYPES OF REINFORCING

diagonal tension failure by use of additional stirrups. Fig. 1 indicates the stirrups used in the various beams. Four of these beams were reinforced longitudinally with hard grade deformed bars, three with square twisted, four with "twin-twisted and stretched," and one beam was reinforced with nickel steel. The stirrups used in all cases were intermediate grade ¼-in. diameter deformed bars with the bamboo or diamond deformations.

The concrete was mixed in 2¼ cu. ft. batches. Each batch was given a three-minute mix. Steel plugs were cast in the compression side of each beam in order to measure the compression strains in the concrete.

At the age of one day the forms were stripped and the beams were placed in the moist room until the age of 28 days at which time they were tested. The specimens were kept damp until they were placed in the testing machine.

Strain readings were taken on both the steel and concrete with a Whittemore strain gage measuring strains to the nearest 1/10,000 in. over a 10-in. gage length. Huggenberger readings were also made on some of the first beams tested but were discontinued because of difficulty encountered in attaching them to the curved surface of a reinforcing bar.

TABLE 1—BEAM DATA

Beam No.	Number and Size of Bars	Type of Bar	Area of Steel sq. in.	Yield Point Stress of Steel p.s.i.	Total Yield Strength of Steel in Beams lbs.	Structural Yield Point Load lbs.	Ultimate Load lbs.	Type of Failure *
C-1	3 ⅜" ϕ def.	Hard Grade	0.324	62 200	20 160	12 500	17 950	T.
C-2	3 ½" ϕ def.	"	0.560	59 200	33 000	19 900	27 750	T.
C-3	3 ⅝" ϕ def.	"	0.890	60 300	53 610	32 400	39 600	T.
C-4	4 ⅝" ϕ def.	"	1.184	60 300	71 480	44 650	51 800	T.
A-1-I	6 ⅜" ϕ def.	"	0.650	62 200	40 300	26 650	30 600	T.
A-1-II	6 ⅜" ϕ def.	"	0.650	62 200	40 300	25 500	30 900	T.
A-2-I	3 ½" ϕ def.	"	0.560	59 200	32 500	21 150	28 650	T.
A-2-II	3 ½" ϕ def.	"	0.560	59 200	32 500	21 100	28 400	T.
A-3-I	2 ⅝" ϕ def.	"	0.590	60 300	35 200	23 300	28 300	T.
A-3-II	2 ⅝" ϕ def.	"	0.590	60 300	35 200	24 400	28 200	T.
B-1-I	6 ½" ϕ def.	"	1.120	59 200	65 000	41 000	45 200	D. T.
B-1-II	6 ½" ϕ def.	"	1.120	59 200	65 000	42 200	45 300	T.
B-2-I	4 ⅝" ϕ def.	"	1.184	60 300	70 900	42 100	44 500	T.
B-2-II	4 ⅝" ϕ def.	"	1.184	60 300	70 900	43 800	45 150	D. T.
B-3-I	2 ⅞" ϕ def.	"	1.193	63 200	75 300	45 050	52 800	T.
B-3-II	2 ⅞" ϕ def.	"	1.193	63 200	75 300	45 800	52 850	D. T.
I-1-I	3 ½" ϕϕ Isteg	Twin Tw. and Stretched	1.180	58 600	69 200	43 800	46 000	D. T.
I-1-II	3 ½" ϕϕ Isteg	"	1.180	58 600	69 200	42 300	45 500	D. T.
T-1-I	5 ½" Sq. tw.	Sq. Twisted	1.250	58 400	73 000	43 500	54 150	D. T.
T-1-II	5 ½" Sq. tw.	"	1.250	58 400	73 000	45 200	54 250	T.
T-2-I	3 ⅝" Sq. tw.	"	1.165	61 800	72 300	44 300	47 100	D. T.
T-2-II	3 ⅝" Sq. tw.	"	1.165	61 800	72 300	43 300	45 700	D. T.
T-3-I	2 ¾" Sq. tw.	"	1.125	64 400	72 100	40 900	42 800	D. T.
T-3-II	2 ¾" Sq. tw.	"	1.125	64 400	72 100	42 100	45 500	D. T.
ST-1	2 ½" Sq. tw.	"	0.500	64 500	32 250	18 200	23 550	T.
ST-2	2 ⅝" Sq. tw.	"	0.780	58 200	45 400	28 900	34 800	T.
ST-3	2 ¾" Sq. tw.	"	1.125	61 000	68 300	42 400	49 400	T.
N-1	2 ⅝" ϕ def.	Nickel	0.615	83 500	50 200	36 700	42 550	T.
IS-1	3 No. 1 Isteg	Twin Tw. and Stretched	0.244	67 800	16 520	13 200	14 700	T.
IS-2	4 No. 2 Isteg	"	0.582	68 000	39 700	27 600	31 150	T.
IS-3	3 ⅝" ϕϕ Isteg	"	0.661	57 200	37 800	26 500	30 300	T.
IS-4	2 ⅝" ϕϕ Isteg	"	1.230	54 000	66 400	46 200	49 600	T.

*T. = tension failure in steel.
D. T. = diagonal tension failure.

Deflections were read on both sides at the center of the beams by means of Ames dials reading to the nearest 1/1000 in. Typical beam deflection curves for each type of reinforcing are shown in Fig. 8.

TEST RESULTS

Tests of Materials—Results of tests of materials have been given in the preceding section and in Table 1.

Typical Tests of Beams—The load-deflection curves in Fig. 8 depict the "load-history" of the beams during three typical tests, giving a graphical picture of all stages of failure. The first break in the curve occurs at loads between 4000 and 12,000 lb. at which time the concrete fails in tension. Cracks show up on the tension side of the beam immediately after this failure and these progress in size and number as the load increases. It should be understood that these cracks are of sufficient size to be plainly visible and are not hair line cracks which are made visible only by soaking in water or through other artificial means. The curve then runs uniformly until the load at which the steel begins to yield. At this point the number of cracks depends

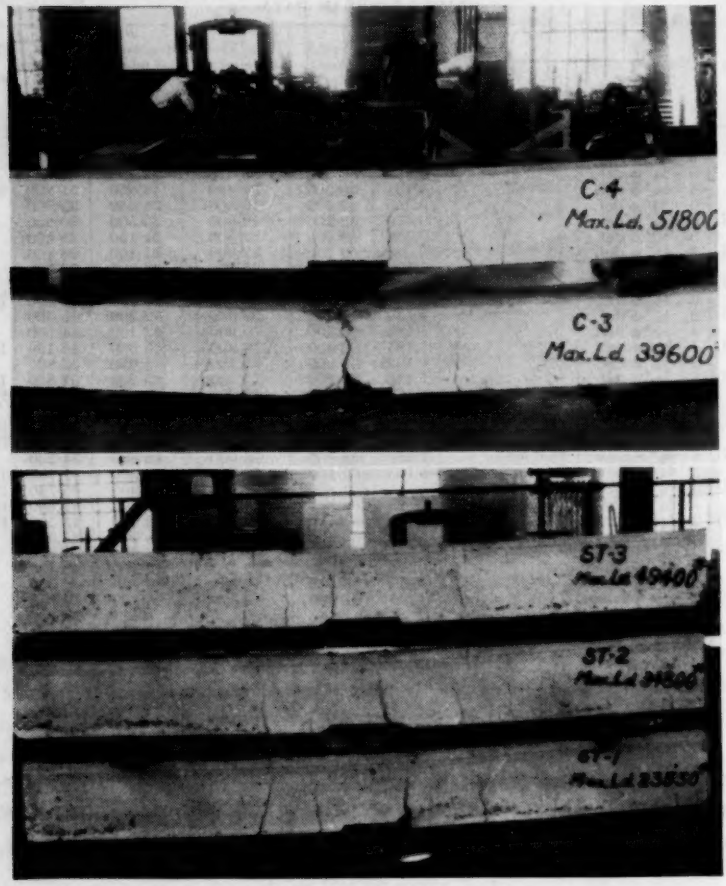


FIG. 9, 10—TYPICAL CONDITION OF BEAMS AFTER THE ULTIMATE LOAD HAD BEEN REACHED FOR BEAMS REINFORCED WITH HARD GRADE DEFORMED BARS AND SQUARE TWISTED BARS, RESPECTIVELY

upon the amount of reinforcing, and for any given number their size depends upon the deflection of the beam. The number of cracks varied from 4 in beams with a low percentage of steel to 18 in the beams with the high percentages. Fig. 9, 10, and 11 show the typical condition of the beams after the ultimate load had been reached for beams reinforced with hard grade deformed bars, square twisted bars, and "twin-twisted and stretched" bars respectively. The upper break in the load-deflection curve will be regarded as the limit of structural usefulness or "structural yield point." The concrete begins to crush shortly after passing the "structural yield point" and the

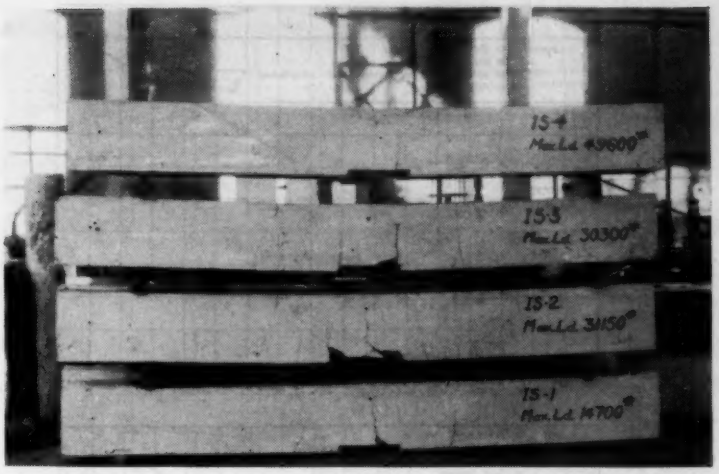
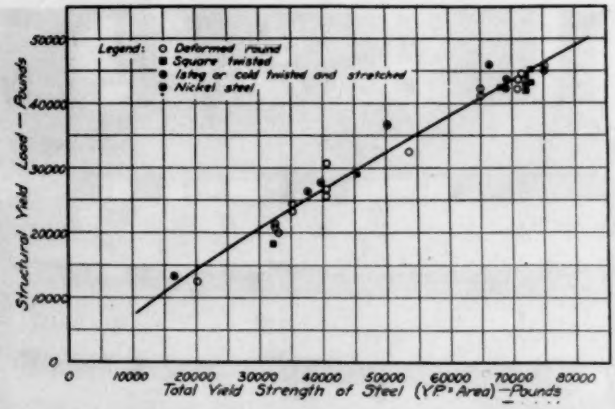


FIG. 11—TYPICAL CONDITION AFTER ULTIMATE LOAD, OF BEAMS REINFORCED WITH "TWIN-TWISTED AND STRETCHED" BARS

TABLE 2—PHYSICAL PROPERTIES OF THE STEEL

Steel	Used in Beams	Method of Obtaining the High Yield Point	No. of Tensile Tests	Yield Point p.s.i.	Ultimate p.s.i.	% Red in Area	% Elong. at 2"	% Elong. at 8"
3/8" ϕ H.Y.P. def.	A-1-I, A-1-II, C-1	High Carbon	12	62 200	92 800	53.8	26.5	17.5
1/2" ϕ H.Y.P. def.	A-2-I, A-2-II, B-1-I, B-1-II, C-2	"	18	59 200	95 100	51.2	38.5	18.3
5/8" ϕ H.Y.P. def.	A-3-I, A-3-II, B-2-I, B-2-II, C-3, C-4	"	12	60 300	93 100	48.4	30.0	18.4
7/8" ϕ H.Y.P. def.	B-3-I, B-3-II	"	4	63 200	106 800	33.9	27.0	15.8
5/8" ϕ H.Y.P. def.	N-1	Nickel	2	83 500	126 900		17.5	8.4
1/2" Sq. twisted	T-1-I, T-1-II	Cold Twisting	10	58 400	70 000	27.2	12.0	5.1
1/2" Sq. twisted	ST-1	"	2	64 500	71 300	29.8	12.5	5.0
5/8" Sq. twisted	T-2-I, T-2-II	"	6	61 800	72 800	31.3	15.5	6.3
5/8" Sq. twisted	ST-2	"	2	58 200	70 000	34.9	16.0	6.5
3/4" Sq. twisted	ST-3	"	2	61 000	70 300	37.7	22.5	9.1
3/4" Sq. twisted	T-3-I, T-3-II	"	4	64 400	75 100	36.5	19.5	7.5
No. 1—Isteg	IS-1	Cold stretching and twisting	3	67 800	85 400	47.2	8.0	2.9
No. 2—Isteg	IS-2	"	4	68 000	79 500	37.4	18.3	
5/8" ϕϕ pl. rd. tw.	IS-3	"	3	57 200	68 800	25.9	12.0	5.4
1/2" ϕϕ pl. rd. tw.	I-1-I, I-1-II	"	3	58 600	72 200	52.8	18.0	6.3
5/8" ϕϕ pl. rd. tw.	IS-4	"	2	54 000	65 800	34.6	20.0	9.1



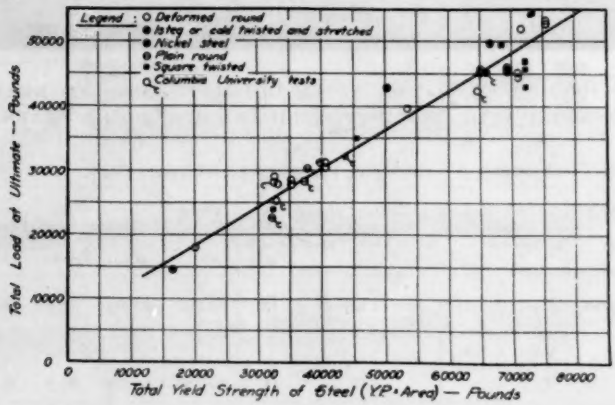


FIG. 12—RELATION BETWEEN STRUCTURAL YIELD AND TOTAL YIELD STRENGTH OF REINFORCING STEEL

FIG. 13—RELATION BETWEEN ULTIMATE STRENGTH OF BEAMS AND TOTAL YIELD STRENGTH OF REINFORCING STEEL

ultimate strength of the beam is quickly reached. The "structural yield point" was arbitrarily determined by the graphical construction shown on the curves in Fig. 8. The construction consisted in bisecting the angle formed by the intersecting extensions of the straight portions of the curve below and above the region of sharp curvature. This method is particularly adapted to the load deflection diagrams corresponding to these beam tests.

SUMMARY OF BEAM TESTS

Fig. 12 presents graphically the relation between total yield-strength of the steel and the structural yield point of the beams. This relation

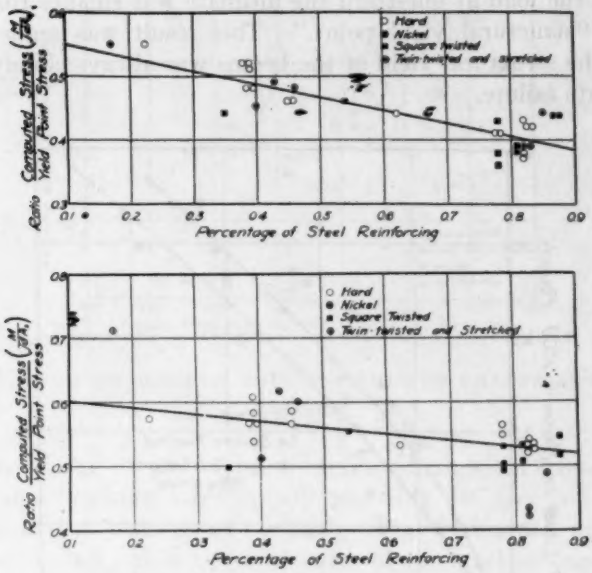


FIG. 14—RATIO OF COMPUTED STRESS AT A BEAM LOAD OF ONE-THIRD ULTIMATE BEAM STRENGTH TO YIELD POINT STRESS OF REINFORCING STEEL

FIG. 15—RATIO OF COMPUTED STRESSES AT BEAM LOAD OF ONE-HALF STRUCTURAL YIELD STRENGTH TO YIELD POINT STRESS OF REINFORCING STEEL

is seen to be nearly linear and is independent of the type of reinforcing steel used.

Fig. 13 shows the relation between total yield-strength of the steel and ultimate strength of the beams. The Columbia tests are included in this diagram and a close agreement is noted with the Lehigh tests. The ultimate strength of the beams is also proportional to the total yield-strength of the steel.

Fig. 12 and 13 show that both the structural yield and ultimate strength of a reinforced concrete beam depend primarily on the total yield-point strength of the steel regardless of the type of bar or manner by which the high yield point is obtained.

Design Loads—Although no definite recommendations will be made in this report as to proper working stresses the test data will be compared at loads of one-third the ultimate and one-half the "structural yield point." Conservative practice would allow the use of the minimum of these two values as a design load. In every beam of the

32 tested the load at one-third the ultimate was smaller than at one-half the "structural yield point." This result was made probable because the structural yield of the beams was always closely followed by ultimate failure.

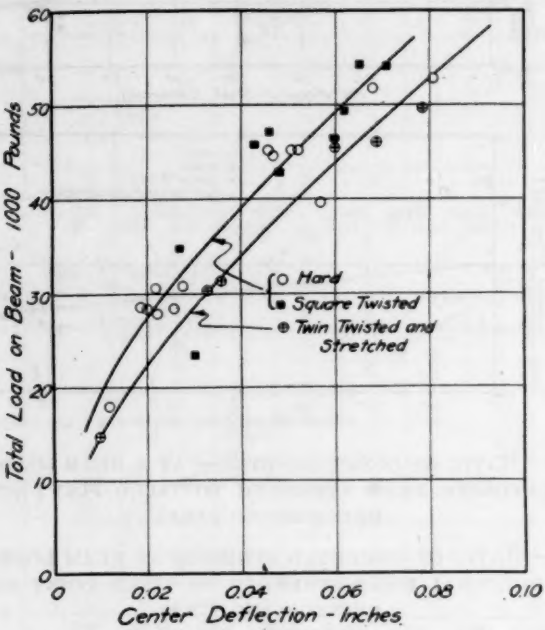


FIG. 16—CENTER DEFLECTION OF BEAMS AT ONE-THIRD ULTIMATE STRENGTH

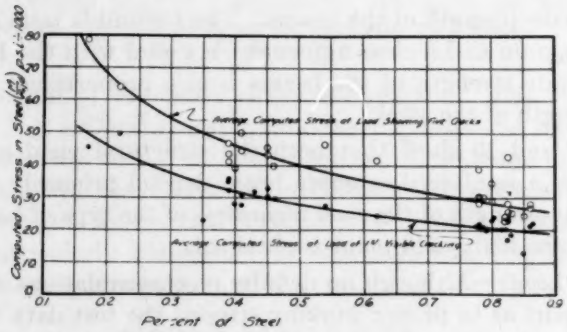


FIG. 17—CURVE SHOWING THE COMPUTED STRESS IN THE STEEL AT FIRST CRACKING AND THE APPEARANCE OF FIVE CRACKS FOR VARIOUS PERCENTAGES OF STEEL

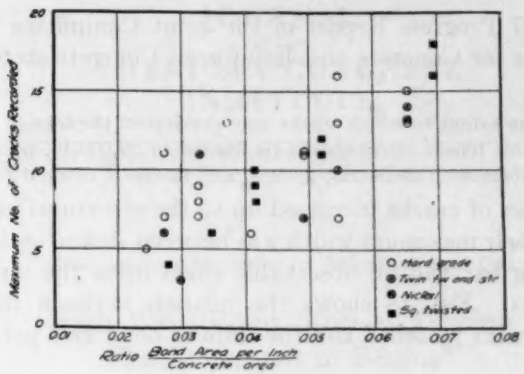


FIG. 18—EFFECT OF BOND ON TOTAL NUMBER OF CRACKS AT ULTIMATE LOAD

If increased stresses are to be allowed for high yield strength steels the allowable working stresses will probably be specified at some percentage of the yield-point strength. Fig. 14 and 15 present the ratio of calculated stress to yield stress of the steel at one-third the ultimate and one-half the structural yield point, respectively. The stress calculation is based on the usual straight-line stress-strain assumption with a value $n = 10$ assumed for straight and square twisted bars and $n = 7.5$ assumed for "twin-twisted and stretched" bars to correspond to a modulus of 22,000,000 p.s.i. The difference between these assumed values of n effects the calculation of stress by only slightly over one per cent.

The deflection of reinforced concrete beams may be a criteria of design in certain cases. Fig. 16 compares the deflections of all the beams at loads of one-third the ultimate strength. The results are somewhat scattered but the average deflection of the beams reinforced with "twin-twisted and stretched" bars ranges from 20 to 35 per cent greater than the average for the beams reinforced with either hard grade or square twisted bars. This increase of deflection agrees well with the fact that the modulus of "twin-twisted and stretched" bars was 25 per cent lower than that of straight bars.

The development of cracks on the tension side of the beam was noted carefully during all the tests. The lower curve in Fig. 17 shows the computed steel stresses for loads at which the first cracks were plainly visible (not hair-line cracks), and the upper curve indicates the computed steel stresses at the appearance of five cracks. For the higher percentages steel the first visible cracks were noted at steel stresses in the neighborhood of 20,000 p.s.i. In this connection the

January 1937 Progress Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete states in Section 875:

In view of the extent to which cracks may develop on the tension face of flexural members the unit tensile stress should be limited to 20,000 lb. p.s.i. in important structural members such as beams, girders, and members of rigid frames.

The number of cracks increased up to the structural yield point, at which load their maximum width was between $\frac{1}{32}$ and $\frac{3}{64}$ in. The type of reinforcing bar had no observable effect upon the number or size of the cracks. Fig. 18 shows the relation between the maximum number of cracks recorded and the ratio of bond area per inch to concrete area.

CONCLUSIONS

1. Both the general "structural yield" and ultimate strength of reinforced concrete beams are proportional to the total yield strength of the tensile reinforcing (yield point stress times steel area) irrespective of the type of bar provided that diagonal tension failure does not occur.

2. No peculiar advantages or disadvantages as tensile reinforcing other than the difference in their respective yield points pertained to any of the types of bars tested except for differences in beam deflection.

3. Beams with "twin-twisted and stretched" bars deflected from 20 to 35 per cent more at a working load of $\frac{1}{3}$ the ultimate than the average of beams with hard grade or square twisted bars.

4. With only three exceptions out of 32 beams tested no cracks were visible to the eye at close range at computed steel stresses under 20,000 p.s.i.

5. Within the range of steel percentages used in the present series of tests (less than 1.00 per cent) the maximum allowable working stresses would be: (a) 40 per cent of the yield-point stress for a factor of safety or 3 with respect to the ultimate strength of the beam; (b) 50 per cent of the yield-point stress for a factor of safety of 2 with respect to the structural yield point.

Discussion, to close in February, 1940 JOURNAL, *should reach A. C. I. Secretary in triplicate by Dec. 1, 1939.*

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Discussion of a paper by Johnston and Cox

High Yield-Point Steel as Tension Reinforcement in Beams*

BY CHARLES S. WHITNEY, D. B. STEINMAN AND THE AUTHORS

BY MR. WHITNEY†

The tests reported in this paper confirm the theory that the flexural strength of a beam depends on the yield point strength of the steel as expressed by the writer's formula[1] for the resisting movement of an under-reinforced beam,

$$\frac{M}{bd^2} = pf_s\left(1 - \frac{pf_s}{1.7\,f'_c}\right) \qquad \text{Eq. (7)}$$

in which f_s is the yield point stress in the steel and f'_c is the cylinder strength of the concrete.

They also indicate the effect of the method of determination of the yield point stress in the steel on the accuracy of the formula.

Table 3 gives the beam load calculated from the writer's formula for comparison with the structural yield point load and the ultimate load as determined by the authors. Fig. 19 shows the calculated load as plotted against the structural yield point load.

Table 4 gives the average ratios of actual to calculated loads for the groups of beams with hard grade, square twisted and Isteg reinforcement. In the case of ultimate loads, those beams failing in diagonal tension were not included in the average.

It appears that the load calculated with the writer's formula agrees very closely with the structural yield point load for the beams reinforced with hard grade and square twisted bars. The beams rein-

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†Consulting Engineer, Milwaukee, Wisc.

[1]Equation (7) page 490, Vol. 33, *Proceedings*, Amer. Concrete Inst. ,"Design of Reinforced Concrete Members Under Flexure or Combined Flexure and Direct Compression", by Charles S. Whitney.

TABLE 3—COMPARISON OF ACTUAL AND CALCULATED BEAM LOADS

Beam No.	Structural Yield Point Load (Lbs.)	Ultimate Load (Lbs.)	Calculated Load (Lbs.) Eq. (7)
C-1	12,500	17,950	13,103
C-2	19,900	27,750	21,070
C-3	32,400	39,600	33,340
C-4	44,650	51,800	43,240
A-1-I	26,650	30,660	25,460
A-1-II	25,500	30,900	25,460
A-2-I	21,150	28,650	20,767
A-2-II	21,100	28,400	20,767
A-3-I	23,300	28,300	22,390
A-3-II	24,400	28,200	22,390
B-1-I	41,000	45,200	39,720
B-1-II	42,200	45,300	39,720
B-2-I	42,100	44,500	43,000
B-2-II	43,800	45,150	43,000
B-3-I	45,050	52,200	45,370
B-3-II	45,800	52,850	45,370
I-1-I	43,800	46,000	42,043
I-1-II	42,300	45,500	42,043
I-1-I	43,500	54,150	44,090
T-1-II	45,200	54,250	44,090
T-2-I	44,300	47,100	43,780
T-2-II	43,300	45,700	43,780
T-3-I	40,900	42,800	43,710
T-3-II	42,100	45,500	43,710
ST-1	18,200	23,550	20,620
ST-2	28,900	34,800	28,527
ST-3	42,420	49,400	41,650
N-1	36,700	42,550	31,400
IS-1	13,200	14,700	10,770
IS-2	27,600	31,150	25,200
IS-3	26,500	30,300	24,000
IS-4	46,200	49,600	41,110

TABLE 4—AVERAGE RATIO OF ACTUAL TO CALCULATED LOADS

Type of Reinforcement	Structural Yield Point Load Divided by Calculated Load	Ultimate Load Divided by Calculated Load
Hard Grade	1.013	1.233
Square Twisted	0.980	1.195
Isteg	1.100	1.268

forced with Isteg bars show about ten per cent higher strength but the discrepancy is obviously due to the manner of determining the yield point stress of the steel.

For use in Eq. (7) the yield point stress in the steel has been defined as the stress producing a total strain of 0.004 inches per inch. It is the total strain in the steel which causes disruption of the concrete beam, not just the plastic strain which is considered in the determination of the yield point by the A.S.T.M. offset method. It would therefore probably be better to define the yield point as the stress producing a definite total strain. Additional tests are needed to show whether or not 0.004 is the best value.

Since stress-strain curves were plotted for the reinforcing steel, the writer hopes that the authors will state what the yield point would be if defined as the stress producing a total strain of 0.004 inches per inch so that the effect of that method can be studied.

The authors state that in the working range the modulus of elasticity of the Isteg bars was not materially affected by imbedment in concrete and they evidently used in their calculations the yield point as determined for unembedded bars. Fig. 6 shows that the total

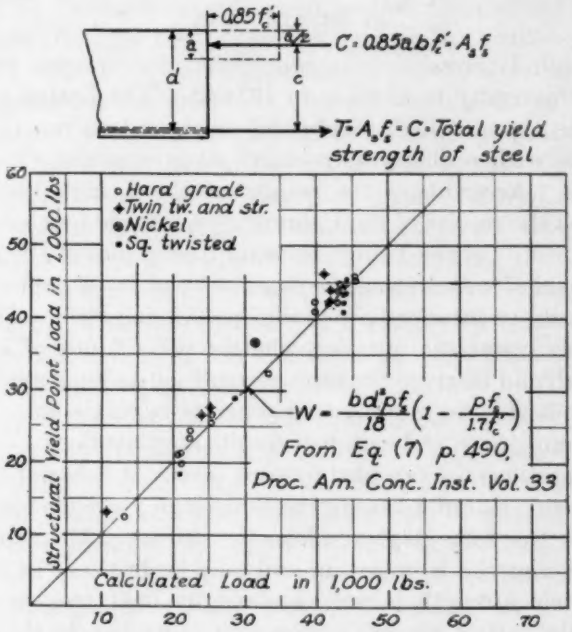

FIG. 19—COMPARISON OF STANDARD YIELD POINT LOADS WITH CALCULATED LOADS

strain was materially reduced by embedment and the yield point of the steel in the beam would be considerably higher than that used. If the yield point determined for embedded bars is used, the agreement between the calculated load and the structural yield point load will be close in the case of the Isteg bars, confirming the Author's conclusion that the strength of the beam depends on the total yield strength of the tensile steel irrespective of the type of bar.

It is interesting to note that the calculated load (by Eq. 7) corresponds so definitely with the value of the observed structural yield point load. It is the writer's opinion that for under-reinforced beams the allowable load should be based on this calculated value instead of the ultimate strength, because the excess strength above this calculated load is due to uncertain factors such as the tension in the concrete and the exact shape of the steel stress-strain curve and it would probably disappear under a comparatively few repetitions of load.

Equation (7) is based on the relations shown in Fig. 19 and is independent of the modular ratio, n. It gives a slightly higher value than the standard formulas based on n.

BY DR. STEINMAN*

The Lehigh University tests supplement but do not parallel the Columbia University tests made in 1934-35. The Lehigh tests cover only various types of high yield-point steel, and do not include any comparisons with ordinary (structural or intermediate grade) reinforcing bars. Accordingly the results of the Lehigh tests do not conflict with the results of the Columbia tests made four years earlier, nor do the results of the Lehigh tests contradict any of the previously recorded conclusions covering the superiority of Isteg steel over ordinary (structural or intermediate grade) reinforcing steel. The present paper merely opens the question whether other kinds of high yield-point steel should be given the same recognition as Isteg steel in higher allowable unit stresses in reinforced concrete beam design.

A mere comparison of beam test results may overlook other important considerations not revealed by such tests. It does not necessarily follow that any material having the same high yield strength as Isteg steel merits the same higher allowable stresses. Heat-treated and high-carbon material is uncertain and may be brittle. In Isteg steel, the high yield strength is not obtained by heat treatment nor by increasing the carbon content. Moreover every bar, in the process of twisting and stretching, is automatically tested for hidden flaws,

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defects, and non-uniformity. In addition, other materials may not have the same high bond resistance as Isteg steel. All of these factors should be given due consideration.

Although six of the test beams in the Lehigh tests were reinforced with "twin-twisted and stretched bars," only two of the beams were reinforced with true Isteg steel; the other four were reinforced with an approximate substitute product that does not show the full advantages of Isteg steel. When the Columbia University tests were made in 1934, true Isteg bars were not yet available in this country and an improvised arrangement had to be used. As recorded in the writer's report (Isteg Steel for Concrete Reinforcement, A. C. I. JOURNAL, Nov.-Dec. 1935, page 183), "It was desired that lengths of 48 ft. be twisted, to minimize the relative effect of longitudinal yield of the machine, but the only machine available was the one used for twisting square bars in which the twisted length was limited to 17 ft. 6 in." The small irreducible minimum of natural yield or play between fixed heads in any machine is relatively more serious as loss of stretch in a bar 17½ ft. long than in one 48 ft. long. Moreover, the twisting heads and clamping means were improvised, so that the product was merely an approximation to the present commercially available perfected Isteg steel. When the Columbia University tests were made, the use of the improvised material was the best available approximation. Those responsible for the tests had to content themselves with the knowledge that those tests would not do full justice to the superior qualities of true Isteg steel. Now that true Isteg steel made in this country is commercially available, there is no longer the same excuse for conducting tests on an improvised approximation.

In order to distinguish the two groups of bars, both of which are designated in the authors' paper as "Twin Twisted and Stretched," they will be referred to in this discussion as "true Isteg" and "improvised Isteg" respectively.

Recent tests made at Columbia University (1939) show that the true Isteg steel as now commercially available has higher physical qualities than the improvised "Isteg." The 1939 tests were on two sizes, .445″$\phi\phi$ and .342″$\phi\phi$, of commercial Isteg bars of intermediate grade. The respective yield strengths were found to be 69,250 and 69,300 p.s.i. respectively, or *6½ per cent higher than the values obtained in the 1934 tests on improvised "Isteg" bars of intermediate grade* (65,000 p.s.i. for ⅜″$\phi\phi$, and 65,000 p.s.i. for ½″$\phi\phi$). The ultimate strengths in the 1939 tests were 83,300 and 82,400 p.s.i. respectively. *When the Isteg bars were encased in concrete, the yield strength was found to be further increased by 7 to 9 per cent, to 74,000 and 76,000 p.s.i. respectively.*

That the true Isteg steel purchased in the open market has higher physical qualities than the improvised "Isteg" that was donated by others is also indicated by a study of the Lehigh test results. According to Table 1 on page 73, the No. 1 and No. 2 Isteg (used in Test Beams No. IS-1 and IS-2) had yield point stresses of 67,800 and 68,000 or an average of 67,900 p.s.i., whereas the improvised "Isteg" ($\frac{1}{2}''\phi\phi$, $\frac{3}{8}''\phi\phi$, and $\frac{5}{8}''\phi\phi$, in Test Beams No. I-1-I, I-1-II, IS-3, and IS-4) had yield point stresses of 58,600, 57,200, and 54,000, or an average of 56,600 p.s.i. In other words, the true Isteg purchased in the open market was more uniform and 20 per cent higher in yield strength than the donated "Isteg" made on an improvised machine. The 20 per cent difference in yield strength is explained as largely due to a difference in the grades of steel used prior to twin-twisting and stretching, but a substantial part of this difference is undoubtedly due to the improvised process of fabrication of the lower value bars. In addition, the beam behavior reveals further superiority of the true Isteg bars. From the same Table 1, page 73, of the Lehigh tests, the following comparison is drawn:

Beam No.	Number and Size of Bars	Total Yield Strength of Steel in Beams Lbs.	Ultimate Load Lbs.	Ratio
I-1-I	3 $\frac{1}{2}''\phi\phi$ "Isteg"	69,200	46,000	.66
I-1-II	3 $\frac{1}{2}''\phi\phi$ "Isteg"	69,200	45,500	.66
IS-3	3 $\frac{3}{8}''\phi\phi$ "Isteg"	37,800	30,300	.80
IS-4	2 $\frac{5}{8}''\phi\phi$ "Isteg"	66,400	49,600	.75
		Average for Improvised "Isteg"		.72
IS-1	3 No. 1 Isteg	16,520	14,700	.89
IS-2	4 No. 2 Isteg	39,700	31, 150	.78
		Average for True Isteg		.84

This comparison shows that the ratio of ultimate load on the beam to the total yield strength of the steel in the beam proved 16 per cent higher (.84 vs. .72) for true Isteg steel than for the improvised "Isteg." This is *in addition to the superiority in yield strength* of the true Isteg over the improvised material. This result is also independently obtained and confirmed from Table 1, page 73, by making a direct comparison of ultimate beam load with total area of steel. This ratio for the 4 beams reinforced with the improvised "Isteg" steel averaged 40,900, and for the 2 beams reinforced with true Isteg steel averaged 56,850, yielding a superior efficiency of 39 per cent for the true Isteg over the improvised "Isteg", although the difference in yield strengths was only 20 per cent. Even if the two beams that failed in diagonal tension are omitted from the comparison, the superior efficiency of

the true Isteg over the improvised "Isteg" is still shown to be 30 per cent. Part of this superiority, but not all of it, may be attributed to the smaller sizes of the true Isteg bars; but it must be remembered that reduction of bar size, with accompanying advantages, is normally identified with Isteg substitution. This also applies to any advantages of reduction of percentages of reinforcement.

A major conclusion of the Lehigh tests is that the general structural yield and ultimate strength of reinforced concrete beams are proportional to the total yield strength of the tensile reinforcing. This confirms and does not controvert the claims made for Isteg steel, and it more than justifies the 50 per cent increase in stress allowed in the New York City Building Code and other codes. The 1934 Columbia tests showed that improvised "Isteg" bars made from intermediate grade billet steel concrete reinforcement rods are 54 per cent higher in yield strength than ordinary intermediate grade bars of equivalent sizes. The 1939 Columbia tests show that true Isteg steel is 6½ per cent higher in yield strength than the improvised "Isteg" bars of intermediate grade. It therefore appears that true Isteg steel of intermediate grade is actually about 64 per cent higher (1.065 x 1.54 = 1.64) in yield strength than the ordinary intermediate grade reinforcing bars which they replace. Hence, according to the major recorded conclusion of the Lehigh tests, a 64 per cent increase in working stress would appear to be justified in the Building Code instead of the 50 per cent actually allowed.

The conclusion that the ultimate strength of reinforced concrete beams is proportional to the total yield strength of the tensile reinforcing irrespective of the type of bar used is only approximately true but is not strictly accurate. The small scale graph on page 76 may not clearly show this, and the failure to discriminate between true Isteg steel and the improvised product further obscures the comparison. The more accurate comparison is obtained directly from Table 1, page 73, by tabulating the ratios of "Ultimate Load" to "Total Yield Strength of Steel in Beams" for the respective individual tests. This comparison (excluding, as the authors have done, the beams that failed in diagonal tension) yields the following results:

RATIO OF ULTIMATE LOAD TO TOTAL YIELD STRENGTH OF STEEL IN BEAMS

Type of Bar	Range of Ratio	Average Ratio
Nickel Steel	—	.85
Isteg Steel	.78 to .89	.83
Hard Grade	.63 to .89	.78
Improvised "Isteg"	.75 to .80	.77
Square Twisted	.72 to .77	.74

The figures in the last column are a measure of relative efficiency beyond mere proportionality to yield strength and are *not* alike for the different types of bar tested. The ratio is lowest (.74) for square twisted bars and highest for Isteg steel (.83) and Nickel steel (.85). The error of indiscriminately confusing the true Isteg steel with the improvised product is also obvious from the above tabulation. The total spread in the last column (.85 vs. .74) is a difference of 15 per cent. This study qualifies Conclusions No. 1 and No. 2 of the authors, and indicates a necessary correction in the interests of complete accuracy. Contrary to Conclusion No. 2 of the authors, the different types of bars tested do reveal relative differences of efficiency as tensile reinforcing other than the mere differences in their respective yield points. With the exception of Nickel steel, which shows a slightly higher ratio, true Isteg steel is at the top of the list.

Insufficient data on the deflection test results are recorded in the paper to permit a full critical comparison. For Fig. 8, page 72—"Typical Load Deflection Diagrams for beams with different types of reinforcing"—three beams are selected for plotting. Despite the selection of three beams not strictly comparable, the one with Hard Grade bars being relatively the heaviest reinforced in terms of total yield strength of steel in beam and the one with improvised "Isteg" being relatively the lightest reinforced in terms of total yield strength of steel in beam, all three beams reached identical deflection *at a high beam load*, namely 48,000 lb. If the three beams had been equally reinforced, with equal total yield strength of steel in beams, and if a beam reinforced with true Isteg were used in the comparison instead of one reinforced with the improvised "Isteg," the comparative results would have been more favorable to Isteg steel.

On the subject of relative deflections, it should be noted that this is seldom a governing consideration in reinforced concrete beam design, since deflections with equivalent steel construction are twice as great and are approved as permissible. Whether the center deflection of a beam at design load is 0.05 inch or 0.06 inch, or any similar ratio, is hardly material in practice. The increasing deflection under overload, between design load and ultimate, may however be more serious in structural consequences; and it is in this region that Isteg steel will resist deflection as well as or better than other comparable hard grade steels, and very much better than ordinary (structural grade and intermediate) reinforcing steel. The increasing deflection under *repeated* load is also of prime structural importance; and for this condition, not covered in the Lehigh tests, Isteg steel has been found by test to be far superior to ordinary steel.

Conclusion No. 5 in the paper (page 80), suggesting maximum allowable working stresses, is based on Fig. 14 and 15 (page 77). Referring to Fig. 14, for a factor of safety of three on the ultimate, it is found that the highest point plotted is that for a beam reinforced with true Isteg steel, and that this point yields an indicated design stress of 56 per cent of the yield point, as compared with the authors' conclusion of a 40 per cent ratio based on the other high yield-point bars. The other true Isteg beam as plotted in the same chart yields an indicated design stress of 45 per cent of the yield point. Some of the beams with square twisted and hard grade reinforcement, as plotted in the same chart, showed ratios falling as low as 36 and 38 per cent. In Fig. 15, for a factor of safety of two on the yield strength, the highest point plotted is again that for a beam reinforced with true Isteg steel; here it is high above the plotted mean graph for the other type beams, and it yields an indicated design stress of 71 per cent of the yield point, as compared with the authors' conclusion of a 50 per cent ratio for all high yield point steel in general. This outstanding showing for Isteg steel is apparently lost from the authors' conclusions because of their failure to discriminate between the true Isteg bars and the improvised "Isteg."

Likewise, in the study of steel stresses at which first cracks appear, the authors have omitted to distinguish the different types of reinforcement. From the diagram, Fig. 17, page 78, one true Isteg point can otherwise be identified in each graph, and this shows a computed steel stress of 45,000 p.s.i. at the first visible crack, and 79,000 p.s.i. at the appearance of five cracks, compared with the cited Joint Committee limitation of 20,000 p.s.i. based on the appearance of cracks with other types of reinforcement.

Referring to the tension tests on twisted bars embedded in concrete, the authors state (page 69): "The concrete spalled off to a greater degree in the 'twin twisted and stretched' bars than in the square twisted bars." No true Isteg bars were included in these tests. It is not stated at what stage of the tests longitudinal cracking and spalling occurred. In the similar tests made at Columbia University on Isteg bars encased in concrete, longitudinal cracks were not detected until the yield point was approached, and spalling did not occur until the ultimate strength was approached. Under these conditions, in view of the high yield point of true Isteg steel (67,800 and 68,000 p.s.i. in the Lehigh tests), the quoted comment by the authors loses practical significance.

For their final conclusions, the authors have excluded the test results on the 10 beams (out of 32 tested) that failed in diagonal

tension, on account of insufficient stirrups to insure against shear failure. On page 71, however, appears the comment: "This indicated a slight tendency toward diagonal failure in the case of beams with the square twisted bars and 'twin twisted and stretched' bars." This observation is misleading and has little signficance, since the beams that failed by diagonal tension were very closely designed to provide about equal strength against shear and bending moment and, under such conditions, chance and minor variations determine which type of failure will occur first and no general conclusions can be drawn. Moreover, the beams that failed in diagonal tension were not strictly comparable for, although they are described as of "nearly equal steel areas," they did not have equal steel areas and they did not have steel of equal yield points. For these 10 beams that failed in diagonal tension, the ratios of ultimate load to total yield strength of steel in beams were as follows:

Hard Grade	—	Range .64 to .70	—	Average .68
"Twin Twisted"	—	Range .66 to .66	—	Average .66
Square Twisted	—	Range .59 to .74	—	Average .65

Obviously, according to this comparison, no significant conclusion can be drawn as to the relative resistance of the different types of reinforcement to failure by diagonal tension. In all fairness, however, it should be noted that the reinforcement designated as "twin twisted and stretched" was the improvised "Isteg", and that no beams reinforced with the true Isteg failed in diagonal tension. This is explained by the extra stirrups used in the final series of twelve tests, but it prevents any conclusions being drawn from the Lehigh tests as to the behavior of true Isteg steel in diagonal tension.

AUTHORS' CLOSURE

The authors appreciate the attention given to their paper by the discussors. Mr. Whitney has used the data in the paper to corroborate his previously published design method. Along the same line the second author, Mr. Cox, has reported similar findings in another paper* as yet unpublished, which presents other test results as well. Mr. Whitney suggests the definition of 0.004 in. per in. for the yield point because he believes that this total strain is more significant in producing disruption of the concrete than the yield point determined by the A. S. T. M. offset method. The application of the term "yield point" to the stress at an arbitrary constant deformation would seem questionable. The term "yield point" implies an increased rate of strain with respect to load, which would not necessarily result from

*"The Balanced Design of Rectangular Reinforced Concrete Beams", Thesis prepared for M. S. degree at Lehigh University by Kenneth C. Cox

the "constant strain limit" of 0.004 in. per in. In the present series of tests the yield points determined by the A. S. T. M. 0.20 per cent offset method are very close to the strain limit of 0.004 in. suggested by Mr. Whitney. This is a natural result of the fact that most of the yield points are in the neighborhood of 60,000 p.s.i. In the case of "true Isteg" No. 1 and No. 2, however, the stress at 0.004 in. strain is somewhat less than the reported yield points. This would not improve the agreement in Mr. Whitney's Table 4, although the use of a yield point obtained by the embedded test would, as he suggests, improve his comparison.

Dr. Steinman states in his discussion that: "Heat-treated and high-carbon material is uncertain and may be brittle" and "In Isteg steel, the high-yield strength is not obtained by heat treatment nor by increasing the carbon content." The authors wish to point out that no heat-treated steels were used in the investigation. In the hard grade bars the carbon content ranged between 0.35 and 0.46 per cent, as reported by the mill, and reference to Table 2 of the published report shows the hard grade material to be of good uniformity with respect to physical properties as well as possessing good ductility.

Starting in the third paragraph and running through the succeeding five paragraphs Dr. Steinman compares in various ways the behavior of "true" Isteg and that of "improvised" Isteg. Although Dr. Steinman states that the difference in yield points and ultimate strength in these two products may be explained as "largely due to a difference in the grades of steel used prior to twin-twisting and stretching" he believes that "a substantial part of this difference is undoubtedly due to the improvised process of fabrication of the lower value bars."

Adopting Dr. Steinman's terminology of "true Isteg" and "improvised Isteg," it should be emphasized that the "true Isteg" was manufactured from what was originally Intermediate Grade steel and the "improvised Isteg" was manufactured from what was originally structural grade reinforcing steel. Even in comparisons of the same grade of steel variations of ten per cent or more might be expected in the yield point as-rolled. If the twisting and stretching process raised the yield point by a constant percentage such variations would be even greater after twisting. In view of this fact any comparison of the relative merits of the "improvised" or "true" twisting processes is incomplete without a statement as to the yield points and ultimate strengths of the straight material prior to twisting.

In the table at the end of paragraph 6 of Dr. Steinman's discussion it should be pointed out that if Beams I-1-I and I-1-II are omitted from the tabulation, because of their diagonal tension failure, the

average ratio of ultimate load to total yield strength will be only 7.7 per cent higher for "true Isteg" than for "improvised Isteg." This is accounted for by the fact that the average percentage of reinforcing steel area was only 0.29 for the "true Isteg" beams as compared with 0.66 for the "improvised Isteg" beams.

The greater relative efficiency of low percentages of steel reinforcing is well known and is verified in the present investigation. Dr. Steinman again makes use of the ratio between ultimate load and total yield strength of the steel. "True Isteg" is again on top of the list which may be explained by the fact that the average percentage of reinforcement was lower than for the other types of steel. The variation in the ratio does not invalidate the straight line relationship between ultimate load and total yield strength but simply means that the straight line does not approach the zero load ordinate for zero percentage of reinforcing.

The conclusion of the authors in respect to deflection is believed to be a conservative estimate. Dr. Steinman states that increased deflection is seldom a governing consideration in reinforced concrete beam design. The authors believe, however, that visible cracking of concrete may be a serious matter in main beams and girders from an external appearance standpoint.

Dr. Steinman cites the high unit stresses reported at the first appearance of cracks and at five cracks for one beam reinforced with true Isteg. This beam has a percentage of reinforcement of only 0.17 per cent. The authors have based their conclusions with regard to cracking on the greater weight of evidence in the region of 0.80 to 0.90 percentage of reinforcing which is closer to the normal design range.

In the report the authors carefully avoided favoring or discriminating against any particular type of reinforcing steel and listed as their major conclusion the fact that the general "structural yield" and ultimate strength of reinforced concrete beams failing in tension are proportional to the total yield strength of the tensile reinforcing irrespective of the type of bar. Dr. Steinman has raised certain questions in regard to the merit of a particular brand of reinforcing steel. The authors have no objection to any particular type of high yield point steel which has good ductility and uniformity. They believe, however, that very careful consideration should be given to the entire problem of design stresses before a general increase in unit stresses is allowed in any steel used as reinforcing in beams and girders.

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Natural Period of Vibration of Building Frames*

By John E. Goldberg†

SYNOPSIS

A method is presented herein for the direct analytical determination of the natural period of simple, free vibration of building frames of the open-panel, shallow-braced type, with particular emphasis on the common and important case of intermediate and moderately tall buildings.

It is shown that the characteristic deflection curve to which the freely vibrating structure oscillates and the dynamic forces which produce the characteristic deflection curve may be determined by a process of successive converging approximations, a process herein introduced as the method of dynamic convergence. In conjunction with the method of dynamic convergence, simplified slope-deflection formulae for the deflection of building frames under lateral load are introduced for use in determining the shape of the deflection curve of the building frame. Having determined the deflection curve of the vibrating frame, the period of vibration is calculated by comparing the potential energy of the deflected structure with its kinetic energy as it passes through its normal static position or figure of rest. The method is demonstrated by solution of a 10-story reinforced concrete building frame.

The basic method of dynamic convergence is applicable to structures and bodies generally, including, among others, trusses, arches, girders, columns, rods, and irregular bodies. The method may be used therefore to determine the natural periods of the various types of bridges, wherein both horizontal and vertical oscillations are important, towers, stacks, machine parts, and other structures and elements.

INTRODUCTION

Determination of the natural period of vibration of building frames is now acknowledged to be a most desirable adjunct to the stress analysis of such bents, particularly in connection with the design of earthquake-resistant structures. The degree with which the natural

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period of vibration of the structure synchronizes with the ground movements may determine whether or not the building will successfully withstand the earthquake. In other than earthquake zones, other questions involving the period of vibration of the building may arise. For example, the decision to house a machine having reciprocating motion or the orientation of such a machine in an existing building, or the design of a new building for such use might well be conditioned by a study of the vibration characteristics of the building. Similar considerations should enter into the design of bridges, smokestacks, towers, and other structures as well as machines and machine parts.

The aim of the author has been to develop a practicable and accurate method of analysis, which, in the case of building frames, is based directly upon and gives due consideration to the unique character of the shallow-braced bent. Since building frames of the open-panel, shallow-braced type are a distinctive structural form having singular deflection characteristics and, hence, singular vibratory characteristics, a method based upon and giving due consideration to these peculiar characteristics possesses an undeniable and inherent advantage over methods based upon comparison with simpler and necessarily unrelated structural forms.

SIMPLE HARMONIC MOTION

Consider a particle having a mass, m, moving with a simple harmonic motion having a period, T, and an amplitude, Δ. As the displacement of the particle from its normal static position approaches the full amplitude, Δ, its acceleration increased uniformly to a maximum value at Δ of

$$\text{acceleration} = \left(\frac{2\pi}{T}\right)^2 \Delta \qquad (1)$$

which, it may be noted, is the centripetal acceleration of a body moving with uniform circular motion. The force with which the particle is moving at the instant it reaches the full displacement is, therefore,

$$\text{force} = \text{mass} \times \text{acceleration} = m\left(\frac{2\pi}{T}\right)^2 \Delta \qquad (2)$$

The work done by the moving particle upon the body or structure which resists or limits the motion of the particle is, since the acceleration and force of the particle have increased uniformly with the displacement,

$$\text{work} = \text{average force} \times \Delta = \frac{m}{2}\left(\frac{2\pi}{T}\right)^2 \Delta^2 \qquad (3)$$

At the instant of passing through its normal static position or figure of rest, the acceleration of the particle is zero, its potential energy having been converted entirely into kinetic energy. Its velocity is

$$\text{velocity} = \frac{2\pi\Delta}{T} \quad \ldots\ldots (4)$$

and its kinetic energy is

$$\text{kinetic energy} = \frac{m}{2}(\text{velocity})^2 = \frac{m}{2}\left(\frac{2\pi\Delta}{T}\right)^2 \quad \ldots\ldots (5)$$

which, of course, is equal to the work done by the particle by the time its displacement has reached the full amplitude, and is identical with the value given by Equation (3).

Considering, now, the bent or frame as a whole, it is clear that the shape of the deflected structure, i. e., the characteristic shape of the deflection curve of the vibrating structure, is determined by the forces exerted upon the structure by the oscillation of the various particles or masses, these forces being evaluated by Equation (2) and, as shown, being proportional to the products of these masses times their respective amplitudes.

A solution may be effected by determining the shape or proportions of the deflection curve to which the structure oscillates and the forces which produce the characteristic deflection curve of the structure, these forces being proportional to the products of the masses times their respective amplitudes, as noted above. It follows, then, that the work done upon the structure by the oscillating masses, which is the potential energy of the deflected structure, may be calculated. By the law of conservation of energy, the kinetic energy of the system as it passes through its figure of rest becomes known. Since the masses are known and the kinetic energy has been determined, the period may be deduced by means of the relation indicated in Equation (5).

DETERMINING THE DEFLECTION CURVE

In determining the shape of the deflection curve to which the structure oscillates under free vibration, use is made of the physical fact that, irregular though the forces may be which initially disturb the structure from its figure of rest, the structure seeks immediately to adjust its figure to the natural deflection curve of free oscillation and with each succeeding oscillation approaches the configuration of the natural curve more closely until, finally, the natural curve is accurately matched and the previous adjusting oscillations are succeeded by self-sustaining free vibration. This fact may be proved experimentally or analytically. We may, therefore, determine the shape of the natural curve by producing analytically a chain of

successive circumstances closely resembling the successive aspects of the structure during its transition from the irregular figure induced by the initial disturbance to the final deflection curve of free oscillation.

Thus, we may begin by assuming an initial deflection curve for the disturbed structure which, for practical reasons, should be as correct an estimate of the final free oscillation curve as we can make conveniently. In the absence of specific data on the exact shape of the free oscillation curve, it will be convenient to assume some simple curve for the initial aspect of the disturbed and deflected structure as, for example, a straight line variation of deflection with height.

The foregoing discussion outlines the general principle of dynamic convergence together with its theoretical and physical background, and also suggests the general method of application of the method of dynamic convergence to structures and bodies. The analytical treatment of the special case of the shallow-braced building frame developed and presented below, together with the illustrative problem will explain the principle and method in greater detail and will indicate by analogy the method of application to other types of structure and bodies.

For the sake of simplicity and convenience in the consideration of the specific case of building frames, it will be assumed that the masses of the several bodies and elements which make up the mass of a story act as a single mass concentrated at the level of the floor system.

By Equation (2), the force exerted by a mass, m, moving with simple harmonic motion is, at the limit of its deflection,

$$\text{Force} = cm\Delta \qquad (6)$$

wherein c is a constant which applies generally to all masses of the system. A form somewhat more convenient for our purpose is obtained by transcribing Equation (6) thusly,

$$\text{Relative Force} = m\Delta \qquad (7)$$

Since each mass moves approximately with simple harmonic motion during the transition or adjusting oscillations, the relative forces exerted by the respective masses may be evaluated by Equation (7). The fact will be appreciated that any minor discrepancy resulting from the divergence between our assumption of simple harmonic motion and the actual motion prevailing during the adjusting oscillations will be absorbed by the same physical process that adjusts the disturbed structure to its natural oscillation curve.

Assuming that each mass has exerted a relative force equal to the product of its mass times its respective initial displacement, as indicated by Equation (7), the first adjusted deflection curve is obtained

for the structure under the action of this group of assumed forces by the use of the following slope-deflection formulae[1] for the deflections of building frames under transverse loads. Each formula is applied in turn to each story of the bent to obtain the adjusted deflection curve.

$$\theta_n = \frac{M_n + M_o}{12\Sigma K_{Gn}} \quad \ldots\ldots (8)$$

$$R_n = \frac{M_n}{6\Sigma K_{Cn}} + \frac{\theta_m + \theta_n}{2} \quad \ldots\ldots (9)$$

Equations (8) and (9) are used for all stories above the first. For the first story, Equations (8a) and (9a) or (8b) and (9b) are used, depending upon the conditions of restraint against rotation at the bases of the first story columns. For the case of full restraint,

$$\theta_1 = \frac{M_1 + M_2}{12\Sigma K_{G1} + \Sigma K_{C1}} \quad \ldots\ldots (8a, \text{ full restraint})$$

$$R_1 = \frac{M_1}{6\Sigma K_{C1}} + \frac{\theta_1}{2} \quad \ldots\ldots (9a, \text{ full restraint})$$

And for the condition of no restraint, i. e., columns assumed to be pin-ended at the base,

$$\theta_1 = \frac{2M_1 + M_2}{12\Sigma K_{G1}} \quad \ldots\ldots (8b, \text{ no restraint})$$

$$R_1 = \frac{2M_1}{3\Sigma K_{C1}} + \theta_1 \quad \ldots\ldots (9b, \text{ no restraint})$$

For all conditions the actual linear deflections are given by

$$D_n = (R_n)(h_n) \quad \ldots\ldots (10)$$

$$\Delta_n = \Sigma_1^n D \quad \ldots\ldots (11)$$

On the basis of the first adjusted deflection curve or transition curve thus determined, a new set of relative forces is computed by means of Equation (7). Again using Equations (8) to (11), a second adjusted deflection curve is obtained. The procedure is repeated until a deflection curve of the desired accuracy is obtained, i. e., until the free oscillation curve is determined with the desired degree of accuracy. The degree of accuracy may be tested by comparison of the successive deflection curves.

[1]For the derivation of the slope-deflection formulae see Appendix II.

Equations (8) to (11) give the deflections due to primary stresses and, therefore, are applicable to low, intermediate, and moderately tall buildings and other buildings of such proportions that the effect of secondary deflections resulting from differential column strains is negligible. For those cases wherein secondary deflections must be considered, a rapid estimate of the secondary corrections may be made using the slope-deflection method developed for the analysis of secondary stresses resulting from wind loads. Similar in character and method of analysis is the correction for the yielding of the foundation soil, a refinement which the engineer may consider in certain important cases. In view the number and complexity of the variables involved and pending the development of this phase of soil mechanics, the engineer must rely upon information obtained at the site in connection with the soil bearing tests. It is suggested that after several loading and unloading cycles are completed, a final load-and-unload cycle should be made covering in uniform increments and decrements the range of, say, from 0.5 to 1.5 times the working unit soil pressure, and an accurate measurement be made of each change in elevation. From the data thus obtained, it will be possible to determine the subsidence or elevation of any footing induced by a given change in the load acting upon that footing. The effect of these changes in elevation of the footings upon the deflections of the superstructure is then analyzed by the slope-deflection method for analyzing secondary effects due to differential column strains.

DETERMINING THE PERIOD OF VIBRATION

Having definitely determined the deflection curve to which the freely oscillating structure deflects and, incidentally, having determined the forces at each story which have induced this deflection, it now becomes possible to deduce a value for the natural period of vibration. At the instant the structure has reached the end of its swing, i. e., is fully deflected, the instantaneous velocity of each particle is zero. The total energy of the system is therefore entirely potential at that instant and is equal to the work done upon the structure by the decellerating particles or, what amounts to the same thing, by the relative forces previously determined. Again, at the instant of passing through its figure of rest or normal static position, the entire energy of the structure is kinetic and is equal to the summation of the kinetic energies of the individual particles or masses of the system. Neglecting the internal losses due to friction and other losses of similar natures, these two quantities of energy must be equal by the law of conservation of energy and as checked by comparison of Equations (3) and (5). Therefore, since

$$\text{Total work} = \Sigma\,(\text{average force } x\,\Delta) = \Sigma\left(\frac{F\Delta}{2}\right) \quad \ldots\ldots (12)$$

wherein, of course, the F and Δ values used are the accepted final and correct values; and since by Equation (5),

$$\text{Total kinetic energy} = \Sigma\left[\frac{m}{2}\left(\frac{2\pi\Delta}{T}\right)^2\right] \quad \ldots\ldots (13)$$

equating (12) and (13) and solving for T gives

$$T = 2\pi\sqrt{\frac{\Sigma(m\Delta^2)}{\Sigma(F\Delta)}} \quad \ldots\ldots (14)$$

It may be noted that where other forms of energy or motion exist in addition to the normal linear form considered, or if a specific loss is to be considered, the necessary corrections may be made to Equations (12) and (13) with consequent modification of Equation (14).

An alternative parallel formula, taking into account the motion of only a single mass, may be obtained by solving Equation (2):

$$T = 2\pi\sqrt{\frac{m\Delta}{F}} \quad \ldots\ldots (15)$$

Being simpler than Equation (14) and entirely satisfactory for normal use, Equation (15) is particularly useful as a means of checking. Equation (14), however, is more easily modified for other than the normal motions and has the further capacity of minimizing irregularities and the errors resulting therefrom.

ILLUSTRATIVE PROBLEM

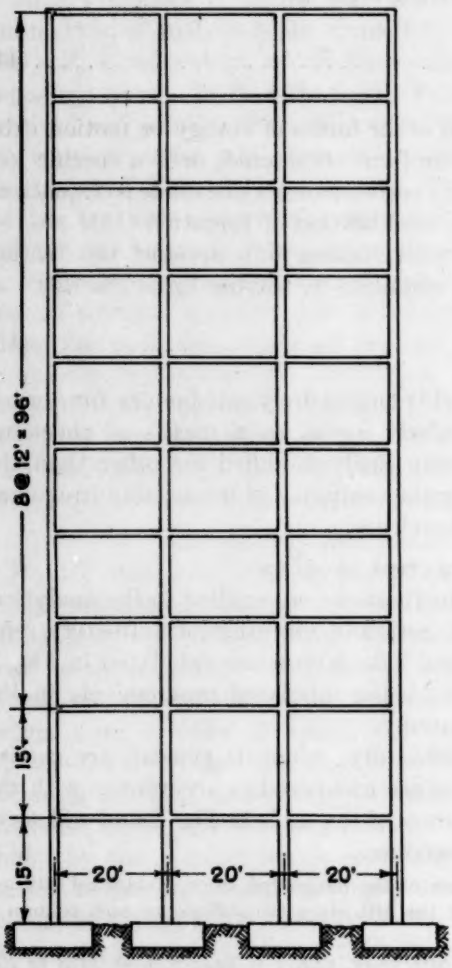
In Fig. 1 the principles set forth above are applied to the analytical determination of the natural period of vibration of a 10-story reinforced concrete bent. Data and calculations are tabulated in Fig. 1, all elements of the calculations being tabulated progressively in the order in which they are computed.

The calculations for the fifth story, which is typical, are carried out in detail below. The items are numbered in accordance with the numbering of the column headings of the table in Fig. 1 and will serve to explain each step of the procedure.

(1) $K_C = 2EI/L$ for each column of the designated story. Tabulated stiffness values are in units of 10^6 lb-ft. At the fifth story the stiffness of each column is $K_{C5} = 49.10$ (x 10^6) lb-ft.

(2) For the four columns of the fifth story, $6\Sigma K_{C5} = 6x4x49.10 = 1178$ ($x\ 10^6$) lb-ft. This item is for use in Equation (9).

(3) As indicated in Fig. 1, the stiffness of each girder of the bent is $K_G = 28.75\ x\ 10^6$ lb-ft. Therefore, for the three girders at each story, $12\Sigma K_G = 12\ x\ 3\ x\ 28.75 = 1035$ ($x10^6$) lb-ft. Note that at the first story $12\Sigma K_{G1} + \Sigma K_{C1} = 1035 + 325 = 1360$



										Data		1st Approx.	
1	2	3	4	5	6	7	8	9	10	Story			
										Kc	10^6 lb-ft	Δ, assumed	ft
81.30	68.80	71.80	59.30	49.10	40.00	32.20	25.65	15.55	8.81	6^2Kc	10^6 lb-ft	Force	lbs
1950	1650	1724	1423	1178	960	772	615	373	212	12^2Kc	10^6 lb-ft	Shear	lbs
1360	1035	1035	1035	1035	1035	1035	1035	1035	1035	Mass		M	lb-ft
4970	4970	4970	4970	4970	4970	4970	4970	4970	3100			$\theta/2$	radians/10^6
										(1)		(5)	
										(2)		(6)	
										(3)		(7)	
										(4)		(8)	
												(9)	
												(10)	$M/6^2Kc$
												(11)	R radians/10^6
												(12)	D ft/10^6
												(13)	Δ ft/10^6
												(14)	Relative Δ
0.119	0.238	0.334	0.429	0.525	0.620	0.715	0.810	0.903	1.000			1.000	
590	1180	1660	2130	2610	3080	3550	4020	4480	3100			3100	
26400	25810	24630	22970	20840	18230	15150	11600	7580	3100			3100	
396000	371150	295500	275500	250000	218600	181800	139000	91000	37200			37200	
282	322	276	254	226	194	155	111	62	18			18	
203	225	171	194	212	228	235	226	244	175			175	
485	829	769	724	692	648	584	492	417	255			255	
7270	12420	9210	8680	8300	7780	7000	5900	5000	3060			3060	
7270	19690	28900	37580	45880	53660	60660	66560	71560	74620			74620	
0.097	0.264	0.387	0.503	0.615	0.720	0.812	0.892	0.958	1.000			1.000	

FIG. 1—ANALYSIS OF A 10-STORY REINFORCED CONCRETE BENT (SEE OPPOSITE PAGE)

2nd Approx.									3rd Approx.									Energy Calculations	
																		$m\Delta^2$	$F\Delta$
lbs	lbs								lbs	lbs									
Force	Shear	lb-ft							Force	Shear	lb-ft								
M		radians/10^6							M		radians/10^6								
$\theta/2$		$M/\theta\Sigma K_C$							$\theta/2$		$M/\theta\Sigma K_C$								
R		radians/10^6							R		radians/10^6								
D		ft/10^6							D		ft/10^6								
Δ		ft/10^6							Δ		ft/10^6								
Relative Δ									Relative Δ										
(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)
3100	3100	37200	18	175	256	3070	81920	1.000	3100	3100	37200	18	175	257	3080	82050	1.000	21.3	257
4750	7850	94100	63	252	431	5170	78850	0.963	4780	7880	94560	64	254	435	5220	79870	0.963	31.6	382
4430	12280	147000	116	239	520	6240	73680	0.900	4470	12350	148200	117	241	525	6300	74650	0.900	27.7	334
4030	16310	195500	165	253	628	7540	67440	0.824	4090	16440	197280	167	256	635	7620	68350	0.824	23.2	280
3580	19890	238500	210	248	706	8470	59900	0.731	3630	20070	240840	212	251	714	8570	60730	0.732	18.3	221
3050	22940	275500	248	234	762	9140	51430	0.628	3120	23190	278280	251	236	771	9250	52160	0.629	13.5	163
2500	25440	305000	280	214	800	9600	42290	0.517	2570	25760	309120	284	217	811	9730	42910	0.518	9.1	110
1920	27360	328000	306	190	862	10340	32690	0.400	1990	27750	333000	310	193	875	10500	33180	0.400	5.5	66
1310	28670	430000	366	261	946	14190	22350	0.273	1350	29100	436500	372	264	960	14400	22680	0.274	2.5	31
480	29150	437500	319	225	544	8160	8160	0.100	500	29600	444000	324	228	552	8280	8280	0.100	0.3	4
																	Totals	153.0	1848

By Equation (15): $T = 2\pi\sqrt{\frac{4970\,(0.058100)}{8180}} = 1.81$ secs.

$T = 2\pi\sqrt{\frac{153.0}{1848}} = \underline{1.81 \text{ secs.}}$

FIG. 1 CONTINUED—(SEE OPPOSITE PAGE)

is tabulated for subsequent use in Equation (8a), whereas the other items in this column are for use in Equation (8).

(4) Mass is the mass assumed to be concentrated at the top, i. e., at the level of the girders, of the story in question and includes the floor system, one-half of the columns, walls, partitions, both above and below the floor level, supported loads, etc. The total weight of these elements divided by the acceleration of gravity is the mass for that story.

(5) For the initial aspect, or estimate, of the deflection curve, a deflection of 1.00 ft. is assumed at the top of the bent with the deflections of all other points in proportion to their elevation. At the fifth story $\Delta_5 = \frac{1.00(66)}{126} = 0.525$ ft.

(6) By Equation (7) the force assumed to be acting at the top of the fifth story is Relative force = $m_5\Delta_5 = 4970(0.525) = 2610$ lb.

(7) The external shear at the fifth story is the total of all the forces from the top of the bent down to and including the force at the top of the fifth story. This item is best computed as a running total: $\text{Shear}_5 = \text{Shear}_6 + \text{Force}_5 = 18\,230 + 2\,610 = 20\,840$ lb.

(8) $M_5 = \text{Shear}_5 \times \text{story height} = 20\,840 \times 12 = 250\,000$ lb-ft.

(9) By Equation (8) $\frac{\Theta_5}{2} = \frac{250\,000 + 218\,600}{2(1035)} = 226.0$ in units of radians/10^6.

(10) $M_5/6\Sigma K_{C5} = 250\,000/1178 = 212.0$ in units of radians/10^6. This item is for subsequent use in Equation (9).

(11) By Equation (9), $R_5 = 212.0 + 254.0 + 226.0 = 692.0$, also in units of radians/10^6.

(12) By Equation (10), $D_5 = 692 \times 12 = 8300$ in units of ft/10^6.

(13) The total deflection of any story, by Equation (11), is the running total of Item (12) from the first story up to and including the story in question. Thus, $\Delta_5 = 37\,580 + 8\,300 = 45\,880$, also in units of ft/$10^6$.

(14) The deflections thus determined are scaled to a value of 1.00 ft. at the top of the bent. Thus, at the fifth story, relative deflection $= \frac{45\,880}{74\,620} = 0.615$. This item is not an essential part of the procedure but is useful chiefly in progressively checking the accuracy of the successive deflection curves. Item (14) is the final item of the first approximation.

(15) to (23) comprise the second approximation or, in other words, determine the shape of the second transition curve. They are a repetition of the processes of (6) to (14) based upon the deflections determined in Item (14). Where Item (14) is omitted, the deflections of Item (13) are used. Comparison of the deflections of (23) with those of (14) shows a change of 2 to 3 per cent, which may be contrasted with the 14 to 18 per cent gain of the first approximation. This indicates, then, that the second approximation corresponds very closely to the final free deflection curve and may, therefore, be used for all ordinary purposes. However, for complete convergence and a final check on the accuracy of the work, a third approximation is made.

(24) to (32) comprise the third approximation which is made in the same manner as the first and second approximation. The perfect agreement between the respective quantities of (23) and (32) will be noted.

(33) The next step in the procedure is to compute the energy quantities for use in Equation (14). Using the actual deflections listed in (31), again taking the fifth story as an example, $m_5(\Delta_5)^2 = 4970(0.052\,160)^2 = 13.5$. $\Sigma(m\Delta^2) = 153.0$ as indicated at the foot of (33).

(34) Correspondingly, $F_5\Delta_5 = 3120(0.052\ 160) = 163$ lb-ft. and $\Sigma(F\Delta) = 1848$.

The final step, calculation of the period by means of Equation (14), is shown at the bottom of Fig. 1. Also shown is an alternative calculation, made by applying Equation (15) to the fifth story. Equation (15) when applied to any or all stories may be used either as the principal formula as a check on the results of Equation (14).

It will be noted that several of the items tabulated in Fig. 1 are merely incidental and, although their tabulation is desirable as an aid to the orderliness of the work, the table may be condensed by omitting these items. A further variation of the general method may be developed by combining Equations (8), (9), (10) and (11) into a single equation having the form, $D_n = aM_m + bM_n + cM_o$.

It may be shown by the methods used in the slope-deflection analysis of wind stresses that the effect of induced differential column strains is to increase the deflections by less than one per cent in the bent of Fig. 1. Since the period varies as the square root of the deflection, the period would be increased by less than one-half per cent. If the effect of relieving stresses is considered, this secondary effect is even further reduced. In general, the secondary effect may be disregarded in all but very tall, slender buildings.

APPLICATION OF THE METHOD

The design of earthquake-resistant structures may be resolved into two primary problems:

(1) The analysis of stresses induced in the structure as a result of the acceleration forced upon the structure by the ground motion; and,

(2) the determination of the natural periods of vibration of the structure, comparison of these periods with the probable period of earthquake motion at the site, together with such modification of the natural periods of vibration as may be necessary to preclude harmonic resonance or synchronism of the free vibration of the building with the ground movement.

Previously, of course, the general static design will have been made.

The analysis of the stresses induced by forced oscillation has been successfully and efficiently based upon rational estimates of the accelerations encountered during earthquakes. Having assumed the acceleration, which is generally taken to be from five to twenty per cent of the acceleration of gravity, depending upon local conditions and the type of structure, the mass of each element or appurtenance of the structure may be invested with the assumed acceleration. The forces acting upon the structure, being simply the respective products of mass times acceleration, thus become known and the stresses induced thereby may be analyzed by the methods used for the analysis of wind stresses.

The method for the analytical determination of the natural period of vibration of building frames presented herein is proposed for the solution of dynamic problems arising in connection with the design of earthquake-resistant structures. Among the most important of these dynamic problems are those concerned with harmonic resonance. Harmonic resonance occurs when the natural period of vibration of the structure and the period of oscillation of the earthquake become synchronized. Under such circumstances the structure may be literally shaken to destruction. Having, on the one hand, seismological data and, on the other, a means of determining the natural period of vibration of the structure under design, the engineer thereby is given a considerable degree of control over the circumstances causing harmonic resonance.

Harmonic resonance may be set up also between a structure and a machine having reciprocating or unbalanced rotating masses. Rarely destructive, the vibration may, however, attain alarming or even serious proportions. In many cases, the structure will have different periods in the direction of each of its major axes, and resonance may be precluded therefore by orientating the machine so that its motion and the natural vibration of the structure in that direction have no harmonic relationship. In other cases circumstances may warrant designing the structure as is done for earthquake resistance, with change in the period of vibration being accomplished when necessary by modifying the rigidity or mass of the structure.

Although presented primarily as a method for the analytical determination of the natural period of vibration of shallow-braced frames, the general method is applicable to trusses, arches, beams, columns, and practically all other structural forms. Both the principle of dynamic convergence and Equation (14) are absolutely general and may be used in conjunction with moment-areas, Williot diagrams, the method of internal or virtual work, or other standard or special deflection = calculating methods to determine the periods of a wide variety of structures including the various types of bridges, towers, stacks, many other ordinary and extraordinary structures, machines, and machine components.

The procedure to use in any case is, briefly:

(1) assume a simple deflection curve, linear, parabolic, etc., depending upon the character of the structure and the distribution of the masses;
(2) calculate the dynamic forces by means of Equation (7);
(3) determine the deflections induced by the assumed dynamic forces of Step (2) using the appropriate deflection-calculating method;
(4) pass the structure through the necessary number of cycles of dynamic conver-

gence, to determine the true free vibration deflection curve, by repeating Steps (2) and (3);
(5) calculate the period by means of Equation (14).

Similarly, when rational methods are used, it becomes possible to include the stiffening effect of walls on the period of vibration of buildings. Where an appreciable percentage of the wall area is occupied by windows or other openings, the action of the wall approximates that of a shallow-braced bent, and the effect of the wall may be determined simply by adding this additional bent to the structural bents. At the other extreme, where the wall presents a sheer unbroken surface with few or no openings therein, it tends to act as a cantilever and may, in the case of low buildings and broad wall areas, exert a controlling effect upon the deflections. This cantilever effect, however, diminishes rapidly and soon becomes negligible as the width of the wall decreases or as the height of the building increases.

APPENDIX I—NOTATION

h = story height, in feet.
m = mass; specifically, the total mass of a story, in slugs.
D = displacement of the upper ends of the columns of a story relative to the lower ends, in feet.
E = modulus of elasticity.
I = moment of inertia of the cross-section of a member.
K = stiffness of a member = $2EI/L$, in lb-ft; K_C = stiffness of a column; K_G = stiffness of a girder or beam.
L = length of a member.
M = product of the external shear acting transversely at a story times the story height, in lb-ft.
R = D/h = angle through which the chord of a column is rotated about an axis normal to the plane of the bent and passing through the lower end of the column, in radians.
T = period of vibration, in seconds.
Θ = rotation of a joint, in radians; Θ_n = an average value of the rotations of the joints at the top of the n-th story, in radians.
Δ = displacement of a mass from its position of rest, also amplitude of vibration, also total deflection of a story, in feet.

For structural purposes, a story consists of the columns of that story together with the girders or beams directly above.

For mass and vibration purposes, a story consists of a floor system together with one-half each of the walls, columns, partitions, etc. above and below that floor plus its appurtenant masses.

Numerical and lower-case subscripts designate the story to which the specific function applies.

APPENDIX II—DERIVATION OF SLOPE-DEFLECTION EQUATIONS

The basic slope-deflection equation for the bending moment at the end, A, of an unloaded girder or beam, AB, is

$$\text{moment} = -K_G(2\Theta_A + \Theta_B) \quad (16)$$

Consider the girders of the n-th story and assume that the rotation of each joint of the n-th story is Θ_n. Then, the moment at each end of any girder becomes $-K_G(3\Theta_n)$, and the sum of the moments at both ends of any girder becomes $-K_G(6\Theta_n)$, and for all girders of the n-th story the sum of the end moments becomes $-\Sigma K_G(6\Theta_n)$.

Assuming that the points of contraflexure of the columns of the n-th story are at the mid-height of the n-th story, the sum of the moments at the top ends of the n-th story columns must be $(\text{shear}_n)\ (h_n/2) = M_n/2$. Similarly, the bending moments at the bottoms of the columns of the o-th story may be assumed to sum up to $M_0/2$. Now, since each joint is in equilibrium, the summation of moments at each joint must be equal to zero and, therefore, the summation of moments at all joints of the n-th story, taken together, must be zero. Thus,

$$\frac{M_n}{2} + \frac{M_0}{2} - \Sigma K_{Gn}(6\Theta_n) = 0 \qquad (17)$$

$$\text{Solving, } \Theta_n = \frac{M_n + M_0}{12\Sigma K_{Gn}} \qquad (8)$$

Again, the basic slope-deflection equation for the bending moment at the top of a column of the n-th story is

$$\text{moment} = K_C(3R_n - 2\Theta_n - \Theta_m) \qquad (18, \text{top})$$

wherein Θ_n and Θ_m are the joint rotations at the top and bottom respectively of that particular column. Similarly, at the bottom,

$$\text{moment} = K_C(3R_n - 2\Theta_m - \Theta_n) \qquad (18a, \text{bottom})$$

The sum of the moments at the top and bottom of that particular column of the n-th story is $K_C(6R_n - 3\Theta_n - 3\Theta_m)$; and, assuming that all joints of the n-th story have the rotation Θ_n while all joints of the m-th story have the rotation Θ_m, the sum of all column end-moments of the n-th story is $(\Sigma K_{Cn})\ (6R_n - 3\Theta_n - 3\Theta_m)$. Since shear is the rate of change of moment, and since $(\text{shear}_n)\ (h_n) = M_n$,

$$M_n = (\Sigma K_{Cn})\ (6R_n - 3\Theta_n - 3\Theta_m) \qquad (19)$$

$$\text{Solving, } R_n = \frac{M_n}{6\Sigma K_{Cn}} + \frac{\Theta_m + \Theta_n}{2} \qquad (9)$$

Where the bases of the first story columns are assumed to be fully restrained against rotation, the sum of the moments at the tops of the first story columns becomes, since $\Theta_o = 0$,

$$\Sigma(\text{moment})_{top} = \Sigma K_{C1}\ (3R_1 - 2\Theta_1) \qquad (20)$$

and for the bottoms of the first story columns,

$$\Sigma(\text{moment})_{bot.} = \Sigma K_{C1}\ (3R_1 - \Theta_1) \qquad (21)$$

Their sum is equal to M_1. Therefore

$$\frac{M_1}{2} = \Sigma K_{C1}\ (3R_1 - 1.5\Theta_1) \qquad (22)$$

From which it will be seen that

$$\Sigma(\text{moment})_{top} = \frac{M_1}{2} - \frac{\Sigma K_{C1}\Theta_1}{2} \qquad (23)$$

Since the sum of the moments around all joints at the top of the first story must be equal to zero,

$$\frac{M_1}{2} - \frac{\Sigma K_{C1}\Theta_1}{2} + \frac{M_2}{2} - \Sigma K_{G1}\ (6\Theta_1) = 0 \qquad (24)$$

Solving,

$$\Theta_1 = \frac{M_1 + M_1}{12\Sigma K_{G1} + \Sigma K_{C1}} \quad \ldots\ldots (8a)$$

Equation (9a) may be derived directly from Equation (9) by substituting $\Theta_a = \Theta_1$ and $\Theta_m = \Theta_o = 0$.

Consider, now, the case wherein the first story columns are unrestrained at their bases, i. e., the points of contraflexure are at the lower ends of the columns. The joints at the top of the first story being in equilibrium, the summation of moments around these joints must be zero:

$$M_1 + \frac{M_2}{2} - \Sigma K_{G1}\,(6\Theta_1) = 0 \quad \ldots\ldots (25)$$

Solving,

$$\Theta_1 = \frac{2M_1 + M_2}{12\Sigma K_{G1}} \quad \ldots\ldots (8b)$$

Equation (9b) may be derived from Equation (9) by inspection if it is remembered that the action of the first story columns with their points of contraflexure at their bases is identical with the action of columns exactly twice as long and having their points of contraflexure exactly at their mid-heights, the upper half of the hypothetical columns being the exact counterpart of the actual first story columns. The effect, then, is to double the effective story height and thereby double the effect of M_1 while halving the effective stiffness of the columns. It will be seen that Equation (8b) may be derived also by the same hypothesis.

Discussion, to close in February, 1940* JOURNAL, *should reach A. C. I. Secretary in triplicate by Dec. 1, 1939.

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Discussion of a Paper by John E. Goldberg

Natural Period of Vibration of Building Frames*

BY M. V. PREGNOFF†

It can be shown that variations in the shape of the deflection curve of a vibrating structure have relatively little effect upon the periods of the structure.

Assume that the deflection curve of the structure will be that caused by its own weight applied laterally. Then the fundamental period of the structure, in seconds, will be

$$T_1 = 2\pi \sqrt{\frac{\Sigma\,(W\Delta^2)}{\Sigma\,(W\Delta)}\,\frac{1}{g}} = 1.1\,\sqrt{\frac{\Sigma\,(W\Delta^2)}{\Sigma\,(W\Delta)}} \qquad \text{.....(14a)}^1$$

where W = weight of structure at each floor, in lbs. (or kips)
Δ = movement of each floor relative to ground, in ft.
g = acceleration due to gravity = 32.2 ft./sec^2

Equation (14a) gives results accurate enough for practical purposes.

In order to determine the movements Δ, the writer developed the following expression:

$$D = \frac{Mh}{3\Sigma C} \qquad \text{.....(26)}$$

where D = distortion within a story; i. e., movement of the top of a story relative to the bottom of the story, in feet.
M = total moment in the story (shear times story height) in lbs. ft. (or kips ft.)
h = story height, in feet.

*JOURNAL Amer. Concrete Institute, Sept. 1939; *Proceedings* Vol. 36, p. 81.
†Hall and Pregnoff, Consulting Engineers, San Francisco.
[1]See "Strength of Materials" by A. Morley, 1913, p. 407.

ΣC = sum of "C" values per story, in lbs. ft (or kips ft.)

$C = K_c \frac{G}{J}$ = elastic constant at each joint of a story in lbs. ft. (or kips ft.)

$K_c = 2E \frac{I}{h}$ = stiffness of column at joint in the story under consideration in lbs. ft. (or kips ft.)

G = sum of stiffnesses $(2E \frac{I}{L})$ of all girders meeting at the joint in question, in lbs. ft. (or kips ft.)

J = sum of stiffnesses of all members meeting at the joint in question, in lbs. ft. (or kips ft.)

The total moment "M" per story is distributed amongst the columns in proportion to "C" values in that story.

For "C" values at the bottom of the columns in the first story use the following:

"C" (at bott. of col.) = 1.5 "C" (at top of col.) when the base of the column is fixed against rotation.

"C" (at bott. of col.) = "C" (at top of col.) when the base of column is partially restrained against rotation.

Equation (26) is derived using the assumptions suggested on page 25 of "Wind Stresses in the Steel Frames of Office Buildings," by W. M. Wilson and G. A. Maney, Bulletin 80, University of Illinois, Eng. Exp. Sta., 1915.

The computations for the fifth story are shown below:

C_1 (at top of exterior column) =

$$49.1 \times 10^3 \times \frac{28.75}{49.1 + 40 + 28.75} = 12.0 \times 10^3$$

C_2 (at bottom of exterior column) =

$$49.1 \times 10^3 \times \frac{28.75}{49.1 + 59.3 + 28.75} = 10.3 \times 10^3$$

C_3 (at top of interior column) =

$$49.1 \times 10^3 \times \frac{28.75 + 28.75}{49.1 + 40 + 28.75 + 28.75} = 19.3 \times 10^3$$

C_4 (at bottom of interior column) =

$$49.1 \times 10^3 \times \frac{28.75 + 28.75}{49.1 + 59.3 + 28.75 + 28.75} = 17.0 \times 10^3$$

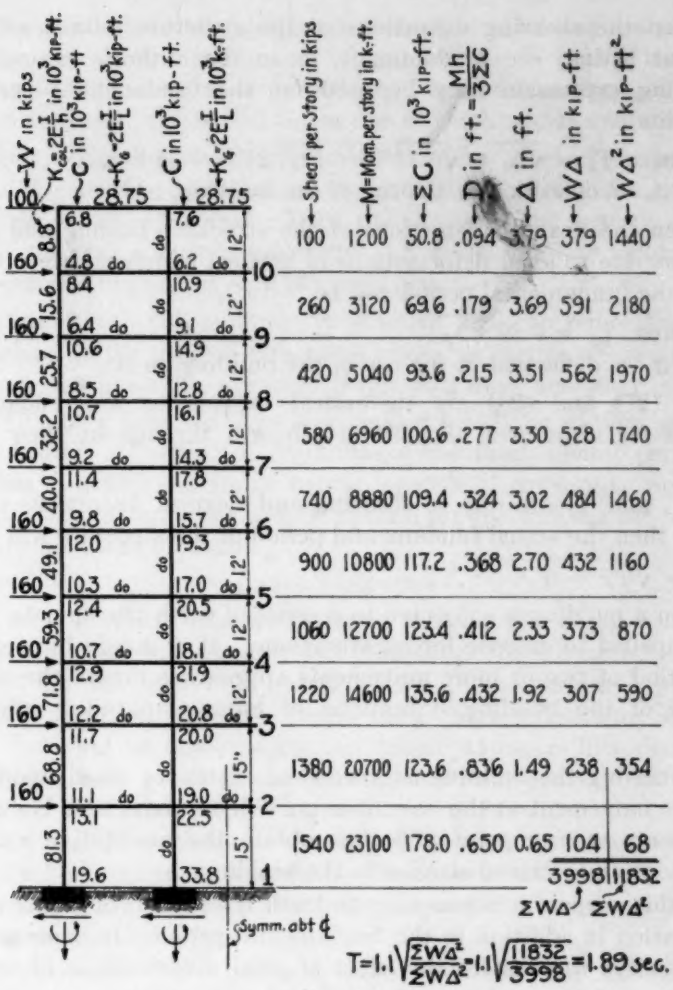

FIG. 2

For ½ Building..... $\frac{1}{2}\Sigma C = 58.6 \times 10^3$

$\Sigma C = 117.2 \times 10^3$

Using eq. (26) $D = \frac{Mh}{3\Sigma C} = \frac{10800 \times 12}{3 \times 117.2 \times 10^3} = 0.368$ ft.

Data and computations are tabulated in Fig. 2.

The fundamental period of vibration is 1.89 seconds when computed by this approximate method. As computed by the author it is 1.81 seconds.

When the shearing distortions of the structure (sliding of floors without tilting) are predominant, as in the author's example, the following expression may be used for the fundamental period of vibration:

$$\text{Approx. } T_s = \sqrt{\Delta_s} = \sqrt{3.79 \text{ (See Fig. 2)}} = 1.94 \text{ Sec.} \quad (27)$$

where Δ_s = deflection at the top of the building, in ft.

When the flexural distortions of the structure (sliding and tilting of floors due to axial deformations of vertical members) are predominant, the fundamental period will be

$$\text{Approx. } T_f = 0.89\sqrt{\Delta_f} \quad (28)$$

where Δ_f = deflection at the top of the building, in ft.

Eq. (27) and (28) are theoretical expressions for fundamental periods for structures of uniform strength throughout their entire height.

If T_s and T_f are due to shearing and flexural distortions respectively, then the actual fundamental period of the structure will be

$$T_1 = \sqrt{T_s^2 + T_f^2} \quad (29)[2]$$

When a building is subjected to a series of earth movements, it will be compelled to execute forced vibrations. If it should happen that the period of two or more movements approaches one of the natural periods of the building, vibrations of large amplitudes may take place.

By treating the building as a vertical cantilever acted upon by a periodic movement at the base, it is possible to arrive at a law of displacements at any point and thus obtain the qualitative range of deflections and induced stresses in the building.

For this purpose it is necessary to know the periods of higher modes of vibration in addition to the fundamental period. In some cases it is imperative to consider the effect of axial deformations of vertical elements and the yielding of the foundation soil upon the natural periods of the building. For a complete treatment of this subject the writer refers the reader to the most excellent paper "Natural Periods of Cantilever Beams" by Lydick S. Jacobsen, Trans. Amer. Soc. Civ. Eng.; 1939, vol. 104.

It has been the writer's experience that in reinforced concrete construction one seldom deals with rectangular frames composed of shallow beams and girders, as far as lateral forces are concerned. In most cases the lateral forces at floor levels are transferred through properly designed floor slabs to the most rigid parts of the building.

[2] "Strength of Materials" by A. Morley, 1913, p. 410.

Usually the exterior walls take most of the forces, leaving a negligible share for the interior beam and column bents. It is this inherant property of reinforced concrete buildings that enables them to resist most economically the lateral forces due to earthquakes and wind.

In computing the periods of the building the effect of the interior bents may generally be neglected, the deflections of the stiffer exterior walls only being considered. Both the shearing and flexural distortions of the wall elements (piers and spandrels) shall be included in the displacements "Δ" of eq. (14) and (14a).

In dealing with the buildings it is worth while to remember that they belong to the type of structures whose properties are vague and uncertain; mathematical accuracy is rarely, if ever, necessary.

Mr. Goldberg deserves to be commended for giving to the engineering profession a method of determining the fundamental period of vibration of a building frame by the process of converging approximations. His method may also be applied for determining the periods of higher modes of vibration.

AUTHOR'S CLOSURE

The most desirable characteristics of a technique or procedure in structural analysis are accuracy and reliability. In the method herein proposed for the analytical determination of the natural period of vibration of various structural forms, these characteristics are assured by the fact that no new or additional assumptions are introduced; in in fact, the only assumptions entering into the analysis are those commonly used and proven assumptions upon which the various deflection-finding methods are based. The accuracy and reliability of the method are further assured by the inherent self-checking nature of methods which employ the principle of iteration or successive converging approximations.

It is axiomatic that a method, even when correctly applied, can be no more accurate than the assumptions upon which it is based. A major source of inaccuracy in the methods hitherto used for the analytical determination of the natural period of vibration is the assumption (which must be made in those methods) of the deflection curve or of the forces which produce the deflection curve. In the method herein proposed, since the final calculations are based upon a true deflection curve and the true dynamic forces which produce that curve, this major error is completely eliminated.

Mr. Pregnoff suggests the use of an approximate method commonly used in machine design work for the approximate determination of the natural period of such parts as shafting, etc. This method

assumes that the forces which produce the natural deflection curve are the weights of the various and several parts of the structure, thereby attempting to reduce the problem to static terms. Actually the dynamic forces which produce the deflection curve are proportional not simply to the various masses but to the product of mass times acceleration of each element of the structure. Take, for example the case of a uniform vertical cantilever. The approximate method would assume that the force per unit of length is constant. Actually the forces increase with respect to height and at a rate which itself increases with the elevation of the element. In the method herein proposed, the dynamic character of the problem is acknowledged.

Vol. 36 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

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Job Problems and Practice

Some will ask questions — Some will answer them — Some will do both

A. C. I. members are invited to use this new JPP department as an informal means toward mutual help. When a problem holds possibilities of general interest the discussion will be briefed in these pages.

If you have a problem—present it; a question, ask it. If you know the answer to a question asked, or if you can contribute something which may help in the solution, or have reason to disagree with the answers or suggestions published, your contribution will be welcome.

If you know of an interesting problem whose solution has already smoothed the way of someone in the field, tell us about it—some other A. C. I. member may need just that information.

The "answers" are the answers of individuals to whom the questions are referred and not of the A. C. I. as an organization.—EDITOR

Out with the "yard" (36-45)

The Kentucky Chapter of the A. I. A. through a special committee is promoting a campaign to abandon the use of the yard as a unit of measure. We are writing to all the professions and associations that should be interested in this subject asking that the use of the square and cubic yard be discontinued by the building profession.

There is no sense in estimating areas and volumes in feet and dividing by 9 or 27 to convert these areas and volumes into units that are awkward and meaningless. Such a change, of course, requires cooperation; nothing can be accomplished singly. It is very like walking around an obstruction in a road. It could very easily be removed by concerted action.

There have been only two objections expressed to the discontinuance of the yard; one is that prices per cubic yard have long been established for concrete, excavation, etc. The other is that the units of square and cubic feet are sometimes too small.

The first objection can be met by the statement that any custom can be abandoned in 24 hours if enough agree to do so. Yard prices can be converted into foot prices in a few minutes.

The second objection can be overcome by using the term "square" (100 sq. ft.) as a unit of area and a "hundred" (100 cu. ft.) as a unit of volume when larger units are desired. Will you give this suggestion your earnest consideration? Your cooperation will be extremely helpful.

Can anyone advance a convincing reason why volumes should be estimated in cubic yards? —Ossian P. Ward

Lead covered cables in concrete? (36-46)

Q—At one of our hydro-plants built about two years ago we have had several failures of lead-covered cables which were installed in galvanized steel conduit buried in the concrete floor between the dam and the power house upstream wall. The failures have been due in each case to corrosion of the lead sheath by the highly alkaline water which finds its way into a few of the conduits and manholes from various joints. As it is not practical to stop the leaks, we have adopted the policy of replacing the cables as they fail with new cables having no lead in their makeup and otherwise not affected by contact with alkaline water. This takes care of the trouble as it develops, but it has not solved the problem as to what precautions to take in future construction with reference to the type of conduit and cable to use and the method of installation to insure long life.

An analysis of the water taken from the conduits gives the following results:

	In parts per million
Total solids (100 deg. C.)	3340
Loss on Ignition	700
Temporary hardness or alkalinity	480
Calcium carbonate, $CaCO_3$	1140
Calcium bicarbonate $Ca(HcO_3)_2$	1850
Magnesium bicarbonate $Mg(HcO_3)_2$	250
Sodium sulphate Na_2SO_4	96
Reaction	Very alkaline

The corrosion products, as determined from analysis of the lead, are hydrated lead monoxide $2PbO.Pb(OH)_2$, lead oxide including PbO, Pb_3O_4, etc. and basic lead carbonate, $2PbCO_3.Pb(OH)_2$.

We are inclined to feel that the conditions at this dam are somewhat unusual as we have experienced no such trouble at other dams, two of which are on the same river. Since they are unusual, we are writing to ask if the American Concrete Institute has conducted any investi-

gations on the effect on galvanized steel conduit, steel pipe, bars, etc., buried in various types of aggregate, i. e. with gravel, blast furnace slag, stone quarried at the site of construction and with either portland or natural cement and if so whether the findings can be made available to us for study? Any help or suggestions you can give us will be appreciated.

A—The foregoing letter to the Institute from an engineer identified with an electric power company was referred to several possible sources of information. From one of them comes these comments and suggestions:

"Our experience has been that leaded cables installed in mass concrete, where moisture exists, are attacked by water containing calcium hydroxide leached from the cement that enters the duct in which the cable may be installed. Particularly, when such cables have been installed inside of dams, care has been taken either to protect the lead sheath thoroughly or to use a leadless type of cable with a jacket little affected by alkaline water. Every effort is made also to drain conduits thoroughly by gravity drain connections.

"Transite conduit has been used to a considerable extent for leaded cable duct lines but care is taken to wrap the joints with tar compound impregnated muslin or jointing compound and all conduits are sloped to drain. Galvanized steel conduit imbedded in mass concrete is avoided where much moisture is expected. Experience on one project, where the soil is sandy and contains alkali, has been that steel pipe is attacked very strongly by these alkaline compounds. In such places where leaded cable is now being installed, transite conduit is being used sloped to drain as mentioned. Non-leaded cables would be selected in such locations except for moisture and high ambient heat difficulties. Kerite type of insulation is successful in such places but the carrying capacity of such cables is usually a limiting factor."

How to design concrete mixtures? (35-21*)

The data presented by Stanton Walker on mix design were reviewed with interest. Usable, simple data on this subject, as Mr. Walker has outlined it, should prove of value to a large number of construction men.

The method described is very similar to that described in Appendix E of the "Concrete Manual" of the Bureau of Reclamation, (Second Edition, March 1939), for obtaining the proportions of an original trial mix involving the use of new aggregate.

*JOURNAL Amer. Concrete Inst., Jan. 1939; *Proceedings* Vol. 35, p. 211; see also June 1939; Vol. 35, p. 584.

The main difference between the two methods is that curves and charts are used in the Manual and tables and computations are used by Mr. Walker. Curves and nomographs developed for this purpose are shown here (Fig. 1 and 2) and there follows a brief outline of the general scheme used.

A start is made from the grading analysis of the sand, (which may be assumed if necessary), to establish the gravel-sand ratio for the maximum size of aggregate to be used. Passing progressively through the other charts, the net water content is determined from which the absolute volume of aggregate can be found for either a fixed water-cement ratio or a fixed cement content. With the absolute volume of aggregate determined, a known or assumed specific gravity will indicate the parts of aggregate by weight. The water-content chart is based on 4-in. slump concrete, which can be modified by the percentage change shown in the note near the bottom of the absolute volume chart.

The Bureau of Reclamation plan for concrete control is based in part upon a maximum water-cement ratio, which, to meet the requirements for density, impermeability, durability, etc., is generally fixed at 0.60 by weight for exposed concrete. For practical purposes the working value must be somewhat lower to provide leeway for the unavoidable variations in the materials and manufacturing processes. A working value of 0.57 or 0.58 by weight has seldom been found to exceed a maximum value of 0.60. Such a water ratio with materials commonly encountered on Bureau projects usually provides more than an ample excess of strength.

It may appear that the method as explained above could be used with acceptable accuracy only when laboratory tests of grading and specific gravity are available. However, assuming the fineness modulus of fine sand at 2.55 to 2.60, or medium sand at 2.75, and of coarse sand at 2.90 to 2.95, will not cause a large error for many types of natural aggregate. Similarly, 2.65 may be used for an average specific gravity if no more exact information is at hand.

It should be emphasized that this scheme is a way to get the very first start to design a mix and that, after the first trial mix has been made either on the job in starting construction or in the laboratory, quantitative, pertinent data will be obtained from which adjustments can readily be made to reach the desired goal.

The adjustment of mixes may be made easily by use of the accompanying nomograph; however, as such charts are not always available the following empirical rules provide a means of quickly computing mix adjustments in the range normally employed:

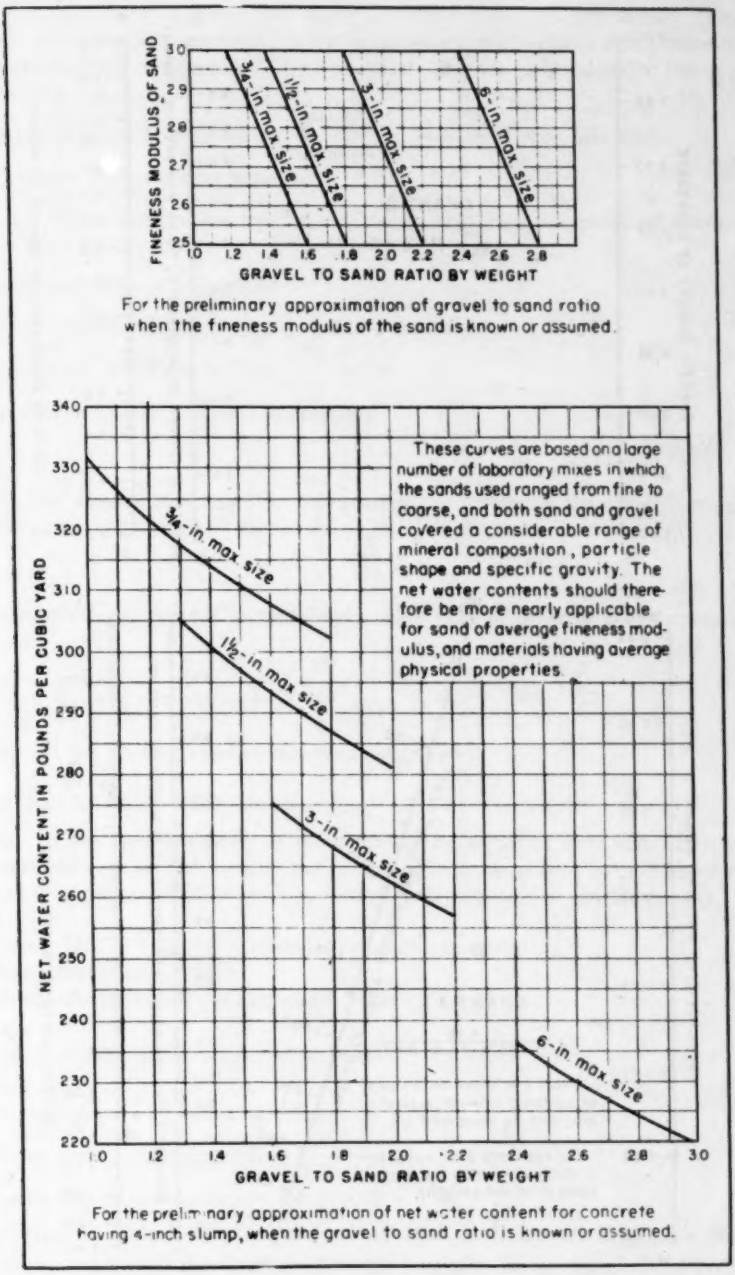
FINENESS MODULUS OF SAND
30
29
28
27
26
25
1.0 1.2 1.4 1.6 1.8 2.0 2.2 2.4 2.6 2.8
GRAVEL TO SAND RATIO BY WEIGHT
¾-in. max size
1½-in. max size
3-in. max size
6-in. max size
For the preliminary approximation of gravel to sand ratio when the fineness modulus of the sand is known or assumed.
NET WATER CONTENT IN POUNDS PER CUBIC YARD
340
330
320
310
300
290
280
270
260
250
240
230
220
1.0 1.2 1.4 1.6 1.8 2.0 2.2 2.4 2.6 2.8 3.0
GRAVEL TO SAND RATIO BY WEIGHT
¾-in. max size
1½-in. max size
3-in. max size
6-in. max size
These curves are based on a large number of laboratory mixes in which the sands used ranged from fine to coarse, and both sand and gravel covered a considerable range of mineral composition, particle shape and specific gravity. The net water contents should therefore be more nearly applicable for sand of average fineness modulus, and materials having average physical properties.
For the preliminary approximation of net water content for concrete having 4-inch slump, when the gravel to sand ratio is known or assumed.

FIG. 1—SELECTION OF NET WATER CONTENT

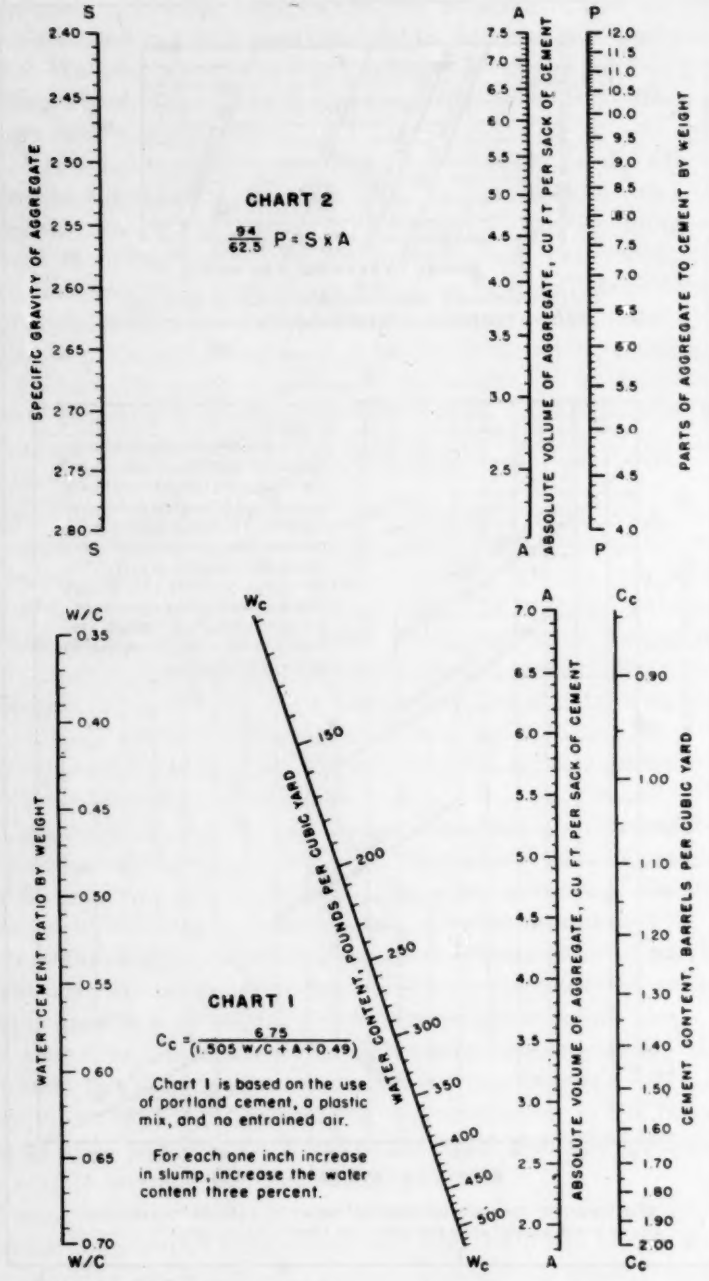

FIG. 2—CHARTS FOR DETERMINATION OF MIX PARTS

Rule 1

For a change in water-cement ratio (the aggregate grading and slump remaining constant), the total mix parts are, for all practical purposes, proportional to the water-cement ratios.

$$\text{(Sum or Original Mix Parts)} \times \frac{\text{New W/C}}{\text{Original W/C}} = \text{Sum of New Mix Parts}$$

Sum of mix parts includes cement.

EXAMPLE: The original mix by weight is 1:2.3:3.7 and the proposed change in W/C is from 0.52 to 0.58.

$$\text{Gravel-sand ratio} = \frac{3.7}{2.3} = 1.61$$

$$\text{Sum of Original Mix Parts} = 1.0 + 2.3 + 3.7 = 7.0$$

$$\text{Sum of New Mix Parts} = 7.0 \times \frac{0.58}{0.52} = 7.8$$

New Mix is 1:6.8 or 1:2.6:4.2 by weight

Rule 2

Each 1-in change in slump (the mix proportions remaining the same) is roughly equivalent to 0.02 change in water-cement ratio, in the same direction.

Rule 3

The unit cement content in barrels per cubic yard of concrete is determined as follows:

$$\text{C.C.} = \frac{F}{\text{(Sum of mix parts)} + \text{W/C}}$$

in which

F = 10.7 for ¾- and 1½-inch maximum aggregates,

and

F = 10.9 for 3-inch maximum aggregate.

Note: If the specific gravity of the combined aggregates differs materially from the average value of 2.65, correct the computed cement content by adding or subtracting 0.02 for each 0.05 increase or decrease, respectively, in specific gravity.

Given:—

Concrete mix = 1:2.3:3.7 (1½-inch maximum aggregate)

Water cement ratio = 0.58

Average specific gravity of aggregates = 2.69

To find:—

Cement content in barrels per cubic yard

Solution:—

Total mix parts = 1 + 2.3 + 3.7 + 0.58 = 7.58

$$\text{Cement content (uncorrected)} = \frac{10.7}{7.58} = 1.41$$

Cement content (corrected) = 1.4 + 0.02 = 1.43

Results obtained by the use of Rule 3 will be suitable for most purposes, inasmuch as the error should rarely be as much as two per cent.

Comparing the above method with that of Mr. Walker, his Table 2 appears to be in good agreement with the water contents from the Bureau of Reclamation "Concrete Manual" (for ¾-and 1½-in. maximum sizes). Table 2 also shows the changes in water required for different slumps, for various maximum sizes of aggregate. Data, now being compiled in connection with the activities of Committee 613, (Recommended Practice for the Design of Concrete Mixes) are in general agreement with a 7.5 per cent decrease in total water required when changing from a 4-to a 2-in. slump as compared to only 4.5 per cent increase for the change from 4-to 6-in. slump. For practical purposes, the "Concrete Manual" used a 3 per cent change in water for each 1 in. change in slump.

Table 3 appears to be out of line with the other parts of Mr. Walker's article. It is realized that if the cement content is increased the percentage of sand may be decreased and the same workability may to some extent be obtained, but this cannot be done without altering the water-cement ratio. Using a fixed water-cement ratio, as the percentage of sand in the mix is increased the amount of water required to produce a given slump must be increased and therefore the cement content must be increased to maintain the water-cement ratio.

The compressive strengths shown by Mr. Walker for various water-cement ratios do not appear to be out of line with values found on a large number of construction projects.—E. N. VIDAL and W. H. PRICE

JOURNAL
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7400 SECOND BOULEVARD, DETROIT, MICHIGAN SEPTEMBER 1939

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Construction of reinforced concrete towers

Betonstein Zeitung, Berlin, No. 12, V. 12, pp. 168-169, June 25, 1939. Reviewed by F. L. Ehase

An Italian method of construction of reinforced concrete towers, developed by A. and G. Ghira in Rome, is described briefly and its advantages from a construction standpoint are indicated.

The building lime specification DIN 1060

Karl Goslich, *Tonindustrie Zeitung*, Vol. 63, No. 49, p. 561. Reviewed by A. U. Theuer

The author here discusses the new terminology adopted for trade usage in the German lime industry and methods of labeling or marking of containers to avoid certain confusions common on the building lot.

Computations in the design of structural frames and continuous beams

P. Landes, *Beton und Eisen*, Vol. 9, No. 38, p. 152 Reviewed by A. U. Theuer

The author compares new methods of analysis of frames, including that of Loser, for vertical loads, Meyer's resistance method, balance of moments method of Cross and Grinter's method.

Restraint and resistance to rotation

A. Geisslreiter, *Beton und Eisen*, Vol. 38, No. 9, p. 154 Reviewed by A. U. Theuer

The two methods stated above are shown in this article to lead to practically identical results. The advantage of the one method as against the other is said to vary from case to case.

Action of aerial bombs on structures

Bautechnische Mitteilungen, Berlin, No. 1, pp. 4-6, Jan. -May 1939. Reviewed by F. L. Ehase

Considerations of the extent of damage to buildings under different structural conditions and of aerial bombing technique are discussed. Diagrams and photographs illustrate some of the points mentioned. The attributes of reinforced concrete make it an ideal type of construction.

Building lime—method of analysis

Tonindustrie Zeitung, Vol. 63, No. 55, p. 608. Reviewed by A. U. Theuer

A tentative standard for the chemical analysis of building limes is presented in detail. Properties to be determined include loss on ignition, CO_2 content, moisture

content, (mechanically held water), soluble hydrochloric acid residue, soluble SiO_2, total of the trioxides (R_2O_3), Fe_2O_3, CaO, MgO, and the combined SO_3. The methods of taking samples and of determining each quantity are described in detail.

The new airport depot in Budapest

VIRGIL BIERBAUER, *Beton und Eisen*, Vol. 38, No. 8, p. 135, Apr. 20, 1939. Reviewed by A. U. THEUER

The paper gives a general description of a recently completed reinforced concrete airport structure in Budapest, Hungary. It consists of a three-story building, circular in plan, and provides hotel and transit accommodations for passengers, as well as office space for the operating personnel. Complete weather observation and broadcasting stations are provided for.

Cracking and deformability of plain and reinforced concrete

C. SCHREYER, *Beton und Eisen*, Vol. 38, Nos. 7 and 8, p. 127, Apr. 5 and 20, 1939.
Reviewed by A. U. THEUER

The author refers to an article published in 1933 giving an analysis of stresses in concrete under compression up to the point of fracture, and a rule for the variation of n. In the two current articles the author extends his method of analysis to cover the case of flexural tensile stresses and cracking in plain and reinforced concrete beams. The method is then put into a form (tables, graphs, etc.) to facilitate its use by the practicing engineer.

A generalization of the moment distribution method

A. FLORIS, *Beton und Eisen*, Vol. 38, No. 10, p. 172. Reviewed by A. U. THEUER

In a brief introduction the author makes the comment that in the U. S. A., the moment distribution method of Hardy Cross has practically supplanted all other methods in the design of statically indeterminate structural members of uniform cross section. This method is then extended by the author to cover the case of beams of variable cross sections. Deformations resulting from rotations, as well as shearing forces, are considered. Applications to illustrate the use of the theory are included.

Comparative tests on the fineness of cements

F. M. LEA. *Journal of the Society of Chemical Industry*, Apr., 1939 Reviewed by P. H. BATES

There are presented the results of the determination of fineness of three different cements by five different laboratories in five different countries. Three turbidimeter methods were used and a method of pipetting the suspension at prescribed times. Three Wagner turbidimeters were used. The results when reduced to similar assumptions are remarkably close. However, since operators presented original results from data obtained on different assumptions, the reported results show wide differences. This shows the necessity of agreement of basic assumptions upon which turbidimeter methods and calculations are based.

Special Cements

ALTON J. BLANK, *Rock Products*, Vol. 42, No. 8, Aug. 1939, p. 55-60. Reviewed by ROY N. YOUNG

The progress in use of materials as plastic binders and wall plaster in masonry construction is traced from clay through lime, hydraulic lime, lume-puzzolana, to mixtures of portland cement and hydrated lime. Demand for special cements during the last decade has resulted in the development and manufacture of special products. Properties and field of usefulness are discussed for such products as the special type

portland cement used in massive concrete dam construction, high SiO_2 puzzolanic types of cements, and masonry cements for brick, stone mortar and stucco. The author concludes that special cements will have their inning, although lime and portland cement will continue to be used.

The decoration of new plaster and cement

H. M. Llewellyn and H. J. Eldridge, Research Assn. of Brit. Paint Color and Varnish Mfgr., Bulletin No. 29, Jan. 1939, pp. 165-188. Reviewed by T. Chisholm

The difficulties encountered when paints and distempers are applied to new plaster and cement are discussed and illustrated with photographs. The causes and results of the various factors affecting the decorated surfaces are investigated in detail under the headings:—Chemical action, loss of adhesion, efflorescence, defects in plaster, and mould growths. Plaster may be tested for chemical action by allowing a sample to set for 24 hours, painting with one coat of white lead linseed oil paint colored with Prussian blue and allowing the sample to stand for a week under a cover. Chemical action will be shown by a bleaching of the blue color.

The requirements of portland cement, lime plaster and various types of calcium sulphate plasters are conveniently summarized and recommendations for their decoration are set up.

How many cements needed?

P. H. Bates, *Rock Products*, Vol. 42, No. 8, Aug. 1939, pp. 51-53. Reviewed by Roy N. Young

Four types of cement are needed, without reference to other cements, such as slag cements, hydraulic, lime, portland, pozzuolanic, high alumina, natural and slag portland cements. Type A cement would replace the present regular cement covered by Federal Specifications SS-C-206, and A.S.T.M. C9-37, with physical requirements the same as the above specifications, and the chemical requirements of SS-C-206. Type B is a High Early Strength cement similar to that now provided for by Federal and A.S.T.M. specifications. Type C, a low heat cement, should meet a heat of hydration requirement of 60 and 70 calories per gram at 7 and 28 days respectively. Type D cement is similar to that now covered by Federal specifications for sulphate resisting cement, SS-C-211. The author emphasizes the need for direct physical tests to indicate the physical properties of cement in preference to chemical limitations. Unfortunately, since a satisfactory physical test to indicate the durability of the cement has not been developed, it becomes necessary to make use of chemical limitations.

Portland cement stabilization applied to foundation construction

F. B. Butler, *Civil Engineering*, Vol. 9, No. 7 (July 1939) pp. 412-413. Reviewed by J. R. Shank

The problem was to construct a new foundation very close to the existing foundation of an exceedingly heavy but meticulously leveled mechanism which could not be disturbed. The new foundation was to be subjected to impacts of tremendous but indeterminate amounts. The soil was a fine dune sand having maximum particles of 2 m.m. diameter.

The procedure determined upon was to excavate to the level of the bottom of the existing foundation and backfill with tamped stabilized soil up to the level of the bottom of the new foundation. The stabilization was accomplished by using a 10 per cent portland cement mixture with about 12 per cent of moisture.

The soil was spread in 6-in. layers and the proper amount of cement was placed

by hand by emptying cement sacks at predetermined places on top. The soil and cement were then mixed by means of a harrow and disc plow, eight passes being used. The water was added by sprinkling during the mixing in amounts to suit the eye. Following the wetting and mixing, the layer was tamped by means of a sheepsfoot roller. The product weighed 115 lbs. per cu. ft. The compressive strengths of cylinders of this material averaged 275 p.s.i. at 7 days; 392 p.s.i. at 14 days and 456 p.s.i. at 28 days. The spread at 28 days was from 250 to 800 p.s.i. A bearing test of 18 tons per sq. ft. on 3 sq. ft. showed a settlement of 0.0011 ft. The old foundation did not move as much as one-half second of an arc during the construction. The actual cost for 4000 cu. yds. of stabilized material was $10,160 or $2.54 per cu. yd.

The general solution of the problem of eccentrically loaded reinforced concrete cross sections (inclined bending with axial loading)

A. ROUSSOPOULOS, *Beton und Eisen*, Vol. 38, No. 5, p. 79 (Mar. 5, 1939) Reviewed by A. U. THEUER

Existing formulas for the determination of the cross sections of reinforced concrete members subjected to eccentric loadings are considered satisfactory so long as the point of application of the axial load lies along a major axis of symmetry. The author develops formulas for the more general cases wherein the cross section does not have an axis of symmetry or the force is applied outside of an axis of symmetry. The mathematical development is somewhat involved but use is made of numerous charts, etc. in arriving at usual design formulas. (The author disregards the tensile

strength of the concrete.) The formulas are confined to rectangular sections and include all five possible cases which might best be illustrated by the diagram above in which the shaded area is under compression.

Institute of Technological research of Sao Paulo

Bulletin 21, Feb., 1939. Reviewed by P. H. BATES

This bulletin presents four papers:

1. The Method for the Mechanical Testing of Cements Adopted by the New Brazilian Specifications
2. Methods of Chemical Analysis of Portland Cements
3. Methods of Chemical Analysis of Gypsum
4. Observations on the Use of the Wagner Turbidimeter

The methods for the chemical analysis of portland cement and gypsum contained nothing novel. The paper on the mechanical testing of cements, according to the new Brazilian specifications, indicates that a plastic mortar is used in the proportion of one of cement to three of sand. The sand is composed of equal fractions retained between the following sieves: 14—16; 16—30; 30—50; 50—100. The mortar is tested in the form of 5 x 10 cm cylinders. The consistency is determined by the flow table and in general is very close to a half liter of water per kilogram of cement. Considerable data are given showing the relation between tests of cement with this mortar and tests of a 1:5 concrete in which the proportions of fine to coarse aggregate

were varied. There is remarkably close relation between the mortar and the concrete strengths.

Concerning the use of the turbidimeter the author brings out a number of the precautions which are recognized in the United States as necessary to follow in the use of the equipment. He also points out certain of the limitations of the apparatus.

A reprint of the first paper cited above is available in English.

Copper-bearing cement

Research and Invention, Col., Ohio, Dec., 1938, Vol. 11, No. 10, p. 2.
From HIGHWAY RESEARCH ABSTRACTS

A new cement, a product of research, recently announced, appears to have possibilities of great usefulness in making "composition" floors, stucco and mortar for tiles and glass brick. Finely divided copper incorporated with "caustic" magnesia (not the dead-burned *MgO*) and mixed with a solution of magnesium chloride produces in the mixture what appears to be a synthetic atacamite, a mineral that has good resistance to weathering. Thus compounded, the product possesses the advantages of flexibility, resilience, and adhesiveness that characterize what are known as Sorel cements (named for the discoverer) without the disadvantages of poor weathering, efflorescence, and incompatibility with lime that detracted from the usefulness of the earlier material. The new copper-bearing oxychloride cement has several other desirable attributes into the bargain.

Strength of the new cement is high, several times that of the portland. The two materials do not compete, however; the copper-bearing cement, because of its higher cost and high heat of setting, is not intended for massive work. It supplements portland cement.

How admirably the new cement supplements and complements other building materials is indicated by a statement of some of its properties and uses. It is not damaged by water. It bonds perfectly with a great many substances; for example, its adhesion to sandblasted glass is more than 100 p.s.i. It can be troweled, brushed, or sprayed into position. (Setting is very rapid, a matter that should be considered.) As a masonry paint it allows the material underneath to "breathe." And it will take almost any kind of filler or aggregate: sawdust, marble, asbestos, or what not.

Plastic flow of concrete

RAYMOND E. DAVIS, University of California, Berkeley. Annual Report, 1937-38, The Engineering Foundation, New York. From HIGHWAY RESEARCH ABSTRACTS

This investigation has been under way since 1926. It now comprises 30 series of tests. During 1938, progress made on several of the projects is reported as follows:

Series 17. *Effect of Type of Cement, Richness of Mix, and Magnitude of Stress Upon Plastic Flow of Mass-Cured Concretes in Tension and Compression.* In this series of tests, in which unit stresses were relatively low, the rate of flow after the age of two years had become very small. These tests are important because they represent the first attempt, on any comprehensive scale, to study the flow of concrete in both tension and compression, at various ages of loading and with the use of various types of cement. At the earlier ages the concrete appears to flow more rapidly in tension than in compression. The early age flow of concretes made with low-heat type cement is greater than with those made with normal portland cement.

Series 25. *Deflection and Fiber Strain in Beams Under Sustained Load.* In this series, tests were made upon nine plain concrete beams, 4 by 6 by 72 in., and upon eighteen 4 by 6 by 12-in. prisms. Ordinary portland and portland puzzolan cements were used.

Series 27. *Effect of Revibration Upon Plastic Flow of Concrete.* Under the theory of plastic flow of concrete, as now conceived, the major part of the deformation is attributed to the seepage or flow of water from the colloidal gel formed by the hydrating cement. If such is the case, a more dense or compact structure of the mass should reduce the plastic flow.

Series 28. *Effect of Sustained Stress on Bond Between Concrete and Steel.* The effect of sustained stress on bond strength was investigated for two cases: (1) that in which the bond stress is sustained

for a considerable time at a value somewhat less than that required to produce initial slip; and (2) that in which there is in the concrete a compressive stress acting normal to the axis of the reinforcing bar. The specimens were 6-in. cubes containing ⅝-in. plain round bars for pull-out tests. The sustained bond stress was 100 p.s.i., and the sustained compressive stress 600 p.s.i. The stresses were sustained by spring loading from seven days to three months, at which later age the pull-out tests were made.

Under the conditions of moist storage used, the bond strengths at either initial slip or maximum load were greater in the specimens subjected to sustained load than in the companion specimens not subjected to previous load.

Series 30. *Plastic Flow of Concrete During the Early Hardening Period.* Most of the tests of plastic flow have been made on fairly well hardened concrete. Loads have usually been applied not earlier than at seven days. In problems involving thermal stresses in concrete and the stresses from drying shrinkage, the flow that takes place during the very early ages, beginning soon after the final set has taken place, may be an important factor in controlling the extent of cracking that may take place. In order to study this, a series of tests has been outlined involving the use of three types of cement, three richnesses of mix, four consistencies of concrete, and six ages of test varying from 12 hours to 7 days. The tests involve not only the flow under constant sustained loads, but also the flow that occurs during the application of load.

Precast road slabs

The Surveyor, Jan. 20, 1939, p. 91. From HIGHWAY RESEARCH ABSTRACTS

An English patent has been issued on an articulated, precast, reinforced concrete roadway slab. Slabs are to be made at a central production plant, close to the source of cement, aggregate, water, etc., and carried to the required location. Slabs may be of any size convenient for handling. To illustrate methods of construction, a slab 10 ft. long, 6 ft. wide, and 6 in. thick will be assumed.

The slabs are cast in wooden forms which are 70 ft. long, 6 ft. wide, and divided into 10 ft. lengths, 6 in. deep. Each slab is further divided into twelve 9 in. transverse beams by means of strips of standard expansion joint material. Strips 2 in. deep are placed across the bottom of the forms at the designated intervals. Over this is placed link chain mesh reinforcing which is kept under tension while the slab is being cast. The division into transverse beams is completed with 4 in. deep strips of joint material. In addition to the mesh reinforcing, rods are placed longitudinally in each transverse beam. After the concrete has set, forms are stripped and slabs stacked for curing.

After curing, slabs are transported to the site of the roadway and placed on a sand cushion which is used to eliminate minor irregularities of the roadbed surface. This sand bed need not be placed very far ahead of the slab laying. Also, one half of a 20 ft. road may be laid and then opened to traffic while the second half is laid.

The slab can also be made 3 or 4 ft. wide for use as a sidewalk. In this case, removable dividers are placed at the intersection of the links in the mesh, which provides a flexible hinge between beams. When ready, the slab may be rolled up on a drum and transported like a roll of wire screen.

Several advantages are claimed for the "Prekast" concrete road; (1) it can be used immediately upon laying, (2) continuity of manufacture under all weather conditions, (3) more satisfactory storage of materials and control of mixtures and quality of concrete, (4) street repairs are made easier by virtue of the ease of removing slab and replacing and no new material is necessary.

Welded shear reinforcing for concrete beams

DEWEY M. MCCAIN, *Civil Engineering*, Vol. 9, No. 7 (July, 1939) pp. 418-421.
Reviewed by J. R. SHANK

Comparative tests were made of beams having 45 degree welded diagonal tension reinforcement with others having vertical stirrups. An effort was made to have the same amount of web reinforcement in both; the spacing of the 45 degree bars was greater by the secant of 45 degrees. Strain-gage measurements were made at a number of places on the web steel in both cases. Deflections were also taken. Time

studies of welding and other observations for the purpose of cost comparisons were made.

No stresses were noted in the stirrups until after cracking occurred, whereas stresses started in the diagonal welded bars at the first loading. After cracking occurred, the stresses in the welded diagonal bars very closely followed those calculated from the usual formula, $f_v = \frac{0.707V's}{A_v jd}$.

The stresses observed in the stirrups varied widely, depending upon the conditions of the crack formations. The variation was about three times that for the welded bars.

The deflections of the beams having diagonal welded bars was from 83 to 92 per cent of those having the stirrups.

Several welding details were tried. That one which gave the best results was one having the diagonal bars welded alternately to one side or the other of the longitudinal bars. A single line of bars welded to the bottoms of the top bars and the top of the bottom bar tended to split the beam at the ultimate strength load.

The welded units will cost less per beam when the following factors are taken into account. The welding is done in the shop, eliminating the costs and uncertainties of field welding. Fabrication in the field is eliminated. Less steel is used than for stirrups because of the horizontal parts and hooks of the stirrups not being necessary.

The welded units materially stiffen the beam and reduce cracking.

Less time is required in the drafting room because no work is necessary to correlate negative with positive sizes as is the case when many bars are bent from one position to the other.

The Hoyer system of steelwire-concrete

A. Kleinlogel, *Beton und Eisen*, Vol. 38, No. 8, p. 141 (Apr. 20, 1939) Reviewed by A. U. Theuer

The author presents a general discussion regarding the materials, characteristics and progress in applications to building construction of "steel-wire-concrete." As developed by Ewald Hoyer, this material is considered by the author as one distinct from ordinary reinforced concrete. Unlike the latter, prestressed steel-wire-concrete units may be cut into any lengths after manufacture without changing the properties of the material; it is almost elastic (small permanent set) and has no tendency to crack under high working stresses, and requires only about 20 per cent of the amount of steel currently used in reinforced concrete.

Concretes having compressive strengths of 12,000 p.s.i. and steels having tensile strengths up to 380,000 p.s.i. are used. The steel is described as a high carbon (0.7 to 1.0 per cent) crucible steel. No alloys are used. The high strengths, and high yield points are obtained by cold working and annealing processes similar to those used in the production of music wire. The diameter of the wires used ranges from 0.02 to 0.08 in. The wires are incorporated in the concrete under extremely high prestresses. (In test beams up to 250,000 p.s.i.). The most peculiar detail is a general absence of any type of end anchorage for the wires. For a steel cross-section of 0.105 sq. in., Hoyer currently uses about 42 1.5 mm diameter wires well distributed over the concrete cross-section. (The matter of bond stresses is discussed in a recent article by Hoyer and Friederich, *Beton und Eisen*, Vol. 38, No. 6, p. 107.)

A general review of laboratory and field tests (both static and dynamic) carried out on this material in the past year follows. Particular stress is laid on the ability

of this material to effect complete recovery even after being subjected to extremely large deflections.

Patents covering this material are held by the Stahlsaiten-Betonwerken, E. Hoyer and Co., K.G., Berlin-Hallensee. A precast unit plant of 1000 cu. m. capacity is now under construction at Hamburg and 500 and 3000 cu m. plants are to be commenced shortly in East Prussia and Berlin. Twenty additional plants are under consideration for large centers of population throughout Germany. Present plans are said to contemplate the complete substitution of this material for both wood and steel beams currently used in residential (apartment) construction.

Diagrams illustrating typical beam cross-sections and methods of use in floor construction are included.

Improving foundation rock for dams

JAMES B. HAYES, *Civil Engineering*, Vol. 9, No. 5 (May, 1939), pp. 309-312.

Reviewed by J. R. SHANK

The grouting of a dam foundation has, in general, two objectives, the elimination or reduction of leakages and the consolidation of the rock to assure a uniform foundation.

A complete geologic record can be of great assistance. The most satisfactory materials to use are cement and water with or without fine sand or rock flour.

Holes are drilled in the region to be grouted and water is pumped into them to determine the consistency of the mix to be used, and to locate leaks, the larger of which must be sealed prior to grouting. Spacing and depths of holes required depend upon the object of the grouting, the height of the dam, and the condition of the rock. For high dams shallow drilling and grouting serves to make a top seal for high pressure deep grouting. Low pressures for shallow holes range from 25 to 100 lb. and high pressures from 500 to 1000 lb. Care must be taken to avoid lifting the rock stratum or otherwise displacing it. It is usually necessary to use indicators referenced to the bottoms of holes deeper than the grout holes to avoid uplift. Grouting may be delayed until after a considerable part of the dam is constructed to obviate any danger of uplift and to get the necessary grout penetration. If the rock is inclined, many of the surface leaks may have to be plugged with wooden wedges, lead wool, or oakum.

Double-acting duplex reciprocating pumps having cylinders with renewable liners, rubber pistons, and rubber valve facings should be used. A duplicate set of pumps should be provided. The cement grouts may vary in water content from 5 parts to 0.6 parts of water to one part of cement. The pumping rate should be kept fairly constant. In deep or high-pressure holes the consistency of the grout should be reduced if the pressure rises, even to the extent of pumping water only, for short periods. The most efficient piping system provides two lines to the hole; one carries the grout from the pump and the second is a return line through a pressure regulating valve, discharging into a sump. At the hole, the side outlet of a tee in the line leads to the hole through a valve and past a pressure gauge. Plugging in the supply line is avoided by keeping the grout circulating. One hole at a time should be grouted.

When flowing water is encountered, if artesian in character, holes should be drilled upstream of the ground flow to relieve the pressure. Water seams may then be plugged with saw-dust or shavings followed by thick grout under low pressure. After this has set, ordinary grouting may proceed.

Sodium silicate followed by calcium chloride, resulting in a solid, has been used for grouting. Other chemical mixtures of like action have been used. Sodium silicate added to the cement grout helps to stop leaks or flows that would wash out ordinary cement mixtures.

Asphalt-latex joint seal

CARL REID (Oklahoma State Highway Commission) From HIGHWAY RESEARCH ABSTRACTS

A study of subgrade soils in Oklahoma and their relation to the behavior of rigid-type surfaces indicated the necessity of preventing leakage of water through the joints in such construction. This could be accomplished by means of a fabricated joint and a joint sealing compound which would provide a complete and permanent bond to the concrete and a long service life.

Asphaltic sealing material in use had not provided a permanent bond with the concrete, and when of suitable consistency for summer use, became brittle and hard in the winter. Patented metal joint and sealing material tried on experimental projects proved to be very expensive and not completely satisfactory.

Data were secured from the California Division of Highways relative to work done there on a combination of asphalt and latex for a joint sealing material. Further experiment produced the following mixture which has been specified for future contracts:

SC-4 Asphaltic Oil	5.475 lb.
38 Per cent Latex	2.431 lb.
Paraffin	0.0395 lb.
Lime	0.0553 lb.
Kremulso	0.0444 lb.

Experience has proven the necessity for closely observing the proportions indicated and the following procedure should be used for best results. It is suggested that the paraffin, lime and Kremulso be mixed together in the proper proportions in advance of the final mixing operation. This blend may be conveniently poured into block molds and allowed to harden in the sizes to be required later.

The SC-4 asphaltic oil is heated in a standard oil-burner kettle to a temperature of 160°-175°F during which time the proper amount of paraffin, lime, Kremulso is added and thoroughly stirred in. The proper amount of this mixture for a 10-gallon batch is then drawn off and placed in a 12-15 gallon drum provided with an oil-burner and a mechanical stirring device. The temperature of the asphalt mixture is then brought up to 175°-200°F and the required portion of latex is added slowly and stirred. When the batch is thoroughly mixed, the resultant is transferred to pouring pots for application to the joint.

A premolded filler 1-in. wide was used for the bottom two-thirds of the joint, leaving 1½ to 2 in. open joint to be filled with the asphalt-latex seal. The joint was poured flush with the pavement surface, stakes being driven at each end of the joint to confine the sealing material. All joints were cleaned before pouring to insure satisfactory bond between concrete and mixture.

Oklahoma's experience indicates that good joint seal will be obtained if the following precautions are taken: (1) thorough cleaning of the concrete sides of the joint; (2) accurate proportioning of materials; (3) uniformity of control of mixing, temperature, and consistency of each batch; (4) pouring of a uniform flush joint; (5) delay of shouldering operations for one week after pouring of joints; and (6) joints should not be poured if temperature falls below 40°F.

Constitution of portland cement clinker

R. H. BOGUE, *Proceedings of the Symposium on the Chemistry of Cements*, Stockholm, Sweden, 1938, 44 pp. From HIGHWAY RESEARCH ABSTRACTS

The advances that have been made during the past 20 years in knowledge of the constitution of portland cement clinker have come about through three well-recognized directions of effort: first, a continued patient study by many investigators of the phase relations of the components of clinker, particularly of the systems which include Fe_2O_3, MgO and the alkalies; second, a development of microscopic and X-ray technique in the identification and measurement of the phases observed; and third, a systematic study of the effects of each constitutent, individually and collectively, on the properties of the cement, including especially such properties as heat of solution and hydration, resistance to sulfate attack, length changes under different conditions of exposure, and rate of development of strength.

During this period the concept of clinker has become established as a mixture of phases definitely determined by the composition and the heat-treatment rather than as a mixture of hypothetically existing oxides. Recognizing the still incomplete information on these phases, their consideration, with prescribed limitations in interpretation, has been an important adjunct in the manufacture, specification and utilization of improved and special cements.

Perhaps of greatest interest at the moment is the information that is being obtained on the glass phase of clinker. Its composition can be calculated in the four-component system CaO-SiO_2-Al_2O_3-Fe_2O_3 and, with certain reservations, in more complex systems. Its nature has been found to be a metastable transition form between the random atomic orientation of a liquid and the periodic and atomic-selective structure of a crystal. Thermal and microscopic methods have been developed for its approximation. The time element in the crystallization of the liquid and lack of information on the course followed during the process precludes the setting up of precise equations for calculating constitution where a known amount of glass is present, but with certain assumptions equations have been developed for this purpose. These must be used with definite reservations in interpretation, but their use, with such reservations, should still further elucidate the relationships between constitution and properties.

The presence of glass may change greatly not only the amounts but the ratios of the crystalline phases in the clinker, and the properties of the cement may be expected to change accordingly. Thus, by way of illustration the solution of magnesia in the glass may decrease or remove the periclase which might otherwise give rise to serious expansions in concrete structures.

The alkali systems are being studied intensively, and already it is apparent that the small percentages in which they usually occur in commercial slurries may still be adequate to produce a profound effect on the percentages of the major constituents in the clinker.

This investigation has led to increased security in the use of portland cement, an enlarged field of service, and knowledge whereby cement may be designed for future purposes.

Properties of cement subjected to high pressures

DR. F. JAGER, *Cement and Lime Manufacture*, Vol. XII, Jan., 1939. From HIGHWAY RESEARCH ABSTRACTS

Numerous investigations have been made in recent years on the effects of temperature on hydraulic binding materials, but the effects of pressure have not been the

subject of much study. The principal types of hydraulic binding materials have been subjected to increasing pressures to a maximum of 256,000 p.s.i. A definite increase in strength of these binding materials was noted as pressures were increased to 128,000 lb., with a falling off of strength after that point.

Pressures were applied by means of a 200,000 pound testing machine. A steel cylinder was filled with the cement being tested, a steel plunger inserted in cylinder and forced down by the testing machine, compressing the cement. By varying the diameter of the cylinder-plunger combination, all pressures in the desired range were obtained. The effect of the pressure was to compact the cement into a block which was apparently very hard, but which could be crumbled quite easily by hand.

It was noted that cements subjected to high pressures underwent a molecular transformation which was greater as the pressure increased and which completely changed the hydraulic properties of the cement. First, the set was accelerated as the pressure increased up to 128,000 p.s.i. Second, the compressive strength of mortars made from compressed cements, tested after two days was doubled for cements subjected to 128,000 p.s.i. pressure. Above that pressure, resulting strengths decreased. The following table summarizes test data for slag cement, two days after hardening of a standard plastic 1:3 mortar.

Pressure applied to cement Lbs. per sq. in.	Compressive strength of Mortar Lbs. per sq. in.
0	904
28,500	1210
57,000	1420
128,000	1830
256,000	1540

Another interesting fact noticed was that the fineness of the cement, measured on a sieve with 10,000 meshes to the sq. cm. increased slightly with pressure, but decreased suddenly at 256,000 p.s.i. At 128,000 p.s.i. material left on the sieve was composed of small grains massed together, but at 256,000 p.s.i., this material was composed almost entirely of grains several millimeters in diameter.

Two theories advanced, and apparently disproved, as explanations for this increase in strength were first, an increase in the temperature of the cement by reason of the work done in compressing the cement, and second, the release of the air in the cement. Theoretical calculations relative to temperature increase indicated an increase of about 20°C, which was greatly in excess of that observed. Cement subjected to a pressure of about 3 mm. of mercury for a period of 8 hours did not show any improvement in mortar strength.

Examination of the powder before and after compression showed that the proportion of very fine particles is greatly increased by the high pressures. This increase in the finer particles results in a large increase in surface area per unit weight of the cement, which would increase the speed of reaction of cement and water. However, an increase in pressure beyond a certain point seems to cause a reconsolidation of particles, resulting in a reversal of this trend.

Determination of humus content in sand and its influence upon concrete

Nils Sundius and Axel Eriksson, *Betong*, 1939, No. 2, p. 65. Reviewed by Bengt Friberg

Humus, the most common contamination in concrete sand, is present in sand together with iron oxide hydrates, aluminiumoxide, silica, and other substances pre-

cipitated from groundwater. A mixture of these oxides and humus forms dark brown films around the sand-grains, and is also present as fine dust. Seven selected sands of varying contamination were selected, their composition determined and 1:4 test cubes prepared and crushed at 7, 28, and 90 days. Humus contaminated water from a peat-bog was used in one test.

The inorganic substances were separated from the sands with a solution containing .2 g. mol. (24.9 g.) acid ammonium oxalate and .075 g. mol. (12.6 g.) neutral ammonium oxalate in 1 l. water, which has small effect on pure sand, but dissolves the inorganic substances precipitated in it and a small part of the humus. Inorganic precipitations varied from .2 per cent to 3.0 per cent.

The humus in the sand was dissolved by a 2 per cent sodium hydroxide solution, after treatment with 1 per cent hydrochloric acid, and the amount determined colorimetrically by comparison with a standard solution of acidum huminicum in 2 per cent sodium hydroxide. Repeated treatments were necessary. Values obtained were comparable provided similar humus preparations were used and on fresh standard samples. The humus content varied from .0006 per cent to 1.22 per cent, each value correct to within about 30 per cent. The humus content in concrete was successfully determined only after the cement was first dissolved with hydrochloric acid. The insoluble remainder was treated with sodium hydroxide solution and gave a good indication of the amount of humus originally present in the sand.

The concrete strength is clearly influenced by a .04 per cent humus content in the sand, and .2 per cent humus content cut the strengths at all ages to less than half of that with the cleanest sand. The humus-containing water, however, appeared to have no deleterious influence on concrete strength.

The chemical influence of humus on the solution in cured concrete was explored on sand-cement-water mixtures in proportions 4:1:1.5. Samples taken at intervals and investigated for solution constituents and nature of solids showed the most conspicuous effect of the humus to be the accelerated precipitation of gypsum. Simultaneously, the alkalies in solution are decreased and the lime increased. The separation of calcium hydroxide crystals is retarded; whereas in pure mixtures they appear in 2-3 hours, with humus present they could not be seen before 4-6 hours, and with highly humic sand they had not appeared even after 9 hours. The crystals were prismatic rather than the usual hexagonal tables. The rapid precipitation of $CaSO_4$ could not be caused by the humus dissolved by the alkalies of the cement. Instead, it has been shown that the solubility of aluminates increases greatly with humus present, and the probable explanation is that the gypsum rapidly precipitates with aluminates as sulphate-aluminate. In addition to the gypsum precipitation, in effect similar to quick set, the reasons for low strength of concrete containing humus are retarded $Ca(OH)_2$ precipitation and poor adherence between the silicate-gel and the films on the sand grains.

Lime-water treatment of humic sands, while considerably decreasing the soluble humus, does not raise the strength of the concrete appreciably. Thorough burning at 340° - 420°C. for over 15 to 45 minutes, while stirring, effectively oxidizes the humus, but shorter burning apparently released humus from fragments of plants in the sand and produced a dangerous increase in humus. Complete burning raised concrete strengths considerably, but even then, due to the inorganic precipitated substances and dust after burning, the strength was about one-fourth less than when pure sand is used.

Tables, microphotographs, and chemical concentration-time diagrams accompany the text.

Floors for industrial purposes

R. Fitzmaurice and F. M. Lea, *Transactions of the Institution of Chemical Engineers*, Vol. 17, 1939. Paper before Joint Meeting of Institution of Chemical Engineers and Institution of Structural Engineers, London, Jan. 26, 1939. Reviewed by J. C. Pearson

This paper, by distinguished investigators from the staff of the Building Research Station, is an authoritative statement of the properties and requirements of floors in industrial buildings. A considerable portion of it deals with concrete floors, as the most common type, and it is that portion to which this review is mainly confined.

Resistance to impact and abrasion are first briefly discussed in which the seriousness of the factory truck problem is presented, and also the related problem of dust nuisance in certain industries. Relative resistance to wear is difficult to determine because tests must take into account both the nature of the traffic and that of the surfacing material. Resistance to chemical attack is shown to be frequently lowered by traffic abrasion. Mineral oils and distillates do not generally attack well cured concrete, but concrete is more or less permeable to the lighter oils. Concrete is readily attacked by the common organic acids of low molecular weight, such as lactic, acetic and tartaric acids. Those of higher molecular weight are commonly components of vegetable and animal oils and fats, and become aggressive in contact with the lime in set portland cement. Aluminous cements are generally more resistant than portland to organic acids when the concentration of the latter is low. This is true also of resistance to sugar solutions, which are particularly aggressive to portland cement concrete when the solutions are hot, or become fermented, or contain acid fruit juices. As for inorganic salts, sodium chloride is the commonest, but it does not attack the concrete chemically; the grave danger lies in corrosion of reinforcement. Sulphates, including thiosulphates and sulphites, also attack portland cement concrete, but aluminous cement has high resistance to them. Surface hardeners of many different types are used; solutions of sodium silicate, aluminum and zinc sulphates, magnesium and zinc fluosilicates; drying oils, such as linseed and tung; carborundum and mixtures of finely divided iron and ammonium chloride incorporated in the wearing course. Drying oils are the most effective of these in reducing dust. None of these materials should be regarded as affording any very marked increase in the resistance to chemical attack in factory floors, except in so far as abrasion resistance is increased.

Special types of factory floors are discussed in some detail. Thus for dairies and cheese factories the attack comes from organic acids, mainly lactic acid and fats, augmented by heavy traffic abrasion. Experience has shown that concrete, either with or without waterproofers or hardeners, will not usually form a durable floor. The best construction appears to be hard acid resistant granitic asphalt mastic with iron grids laid to form traffic paths. Timber floors might be suitable with provision for renewing the wearing surface economically, but they are likely to become slippery and unsanitary. In meat and food factories, floors are subject to the action of fats, brine and water, combined with heavy abrasion. Quarry tiles with thin neat portland or aluminous cement joints have given satisfactory service. In factories handling animal and vegetable oils the problem is similar, and tiles or hard bricks with thin cement joints are probably the best bet. In factories handling sugar and fruit juices, hard asphalts with inert fillers are sometimes used, but such floors tend to soften and must be kept very well washed. Timber floors with some sealing treatment might be satisfactory if the traffic is light. In tanning and other factories where salt water is freely used, it is essential to cover the structural floor with an impermeable surfacing, such as tar or asphalt composition, and in turn to cover this

with a suitable wearing surface. Hard dense timber may be used satisfactorily for such floors, provided they are not subject to hard wear.

Other phases of the subject are covered in the paper, such as appearance, strength, sound transmission, comfort and volume changes. In connection with the latter the importance of drying shrinkage in concrete floors is dwelt upon at some length. If a jointless wearing surface is required to be monolithic with the structural support in order to retain liquids, it is particularly important that adequate reinforcement should be provided at the top of the structural slab in both directions so as to restrain shrinkage. If it is necessary to provide an impervious wearing surface, and there is any doubt as to restraint of shrinkage cracking, an asphalt mastic should be used as a damp-proof course, capable of conforming to movement of the supporting system without itself cracking.

Effect of sand on the durability of mortar

BERT MYERS (Engineer of Materials and Tests, Iowa State Highway Commission) Highway Research Board. From HIGHWAY RESEARCH ABSTRACTS

The fact that laboratory freezing and thawing approximates certain natural conditions destructive to concrete and rocks recommends it as a means for studying the durability of concrete and concrete materials. In 1936, the Highway Research Board Committee on Durability of Concrete as Affected by Cement reported on the results obtained from a series of mortar specimens subjected to alternate freezing and thawing. The kind of cement used was varied, while all other factors were held constant. It was therefore logical to assume that differences in the effect of this treatment were due entirely to the variation of the cement, and that the results obtained were a measure of the relative durability of the cements used.

By the same reasoning it seems logical to assume that if a series of mortar specimens, in which all factors except the fine aggregate are held constant, is subjected to alternated freezing and thawing, the effects of such treatment will be due entirely to the differences of the aggregate, and the results will be a measure of the relative durability of the aggregate used. To study the reliability of these assumptions, the Iowa State Highway Commission made a series of these tests. Plastic mortars were made from three cements commonly used in Iowa, each combined with five kinds of fine aggregate. One part of cement was used to two parts of aggregate by weight. Specimens were made in a water-tight metal mold, 2 x 2 x 14 in.

The aggregates used varied widely in mineralogical composition as follows:

Sand O. Graded silica sand from Ottawa, Illinois.

Sand P. Platte River Sand from Nebraska, derived from disintegrated granite, transported a great distance by water. Particles retained on No. 16 sieve are 45 per cent quartz, 27 per cent feldspar, 22 per cent granite and small percentages of basic rock and chert.

Sand B. From an old Mississippi River terrace at Bellevue, Iowa, derived from disintegration of rocks. Particles retained on No. 16 sieve are 24 per cent quartz, 8 per cent feldspar, 21 per cent granite 34 per cent basic rock, 6 per cent felsite and small percentages of chert, limestone, sandstone and quartzite.

Sand M. From glacial terrace at Mason City, Iowa. Particles retained on No. 16 sieve are 10 per cent quartz, 35 per cent limestone, 16 per cent feldspar, 21 per cent granite, 13 per cent basic rock and small percentages of other rocks.

Limestone Screenings L. Screenings from quarry at Fort Dodge, Iowa.

The finer material of all these sands contained 65 to 90 per cent quartz particles.

At the end of the 50 cycles the specimens were tested as beams, 12 in. between bearing points, loaded at the third points, and as modified cubes in compression. Resulting strengths were compared with similar specimens of the same age that were not subjected to the cycles of freezing and thawing. The strength of the frozen-

thawed specimens is expressed in Table 1 as a percentage of similar specimens not frozen and thawed.

TABLE 1

Cement	B					F					G				
Aggregate	O	P	B	M	L	O	P	B	M	L	O	P	B	M	L
Transverse	68	39	61	57	78	67	67	83	80	82	73	77	74	67	76
Compressive	81	60	80	74	91	82	77	83	81	84	89	74	74	77	95

It is interesting to note the results of soundness tests of the aggregate conducted according to A.S.T.M. Method C88-37T with sodium sulphate and magnesium sulphate and by freezing and thawing according to the method proposed in 1938 by A.S.T.M. Committee C-9 as given in Table 2.

TABLE 2—LOSS IN WEIGHT, PERCENTAGE

Aggregate	Sodium Sulphate 5 Cycles	Magnesium Sulphate 5 Cycles	Freezing and Thawing 16 Cycles
Ottawa.....O	0.195	0.524	0.364
Platte River.....P	0.622	3.615	1.304
Bellevue.....B	1.350	1.897	1.691
Mason City.....M	2.311	7.074	3.450
Limestone Screenings L	4.530	13.650	16.300

The fact that limestone screenings which produced mortar having the least loss in strength due to freezing and thawing showed the greatest loss in each of the soundness tests suggests that perhaps these methods of soundness test do not give comparable results between rounded particles and extremely angular particles.

The average loss in weight of the specimens during freezing and thawing, indicating the extent of surface disintegration, expressed in percentage of original weight, is given in Table 3.

TABLE 3—LOSS IN WEIGHT FROM FREEZING AND THAWING

Aggregate	O	P	B	M	L
Cement B	2.01	1.00	1.32	0.92	1.50
Cement F	3.08	1.72	2.03	1.56	1.87
Cement G	2.51	1.43	1.89	1.42	1.68

It will be noted that the order of the results from lowest to highest is M, P, L, B, O for cements F and G, while the order for Cement B is the same, except that the positions of aggregates B and L are reversed. The order for cements from lowest to highest is always B, G, F.

The linear coefficient of expansion for each aggregate per degree for the range O F. to 70F. is given in Table 4.

TABLE 4

Aggregate		Coefficient of Expansion
Ottawa	O	.00000913
Platte River	P	.00000870
Mason City	M	.00000793
Limestone	L	.00000770
Bellevue	B	.00000757

When the expansion coefficient is plotted against loss of weight due to freezing and thawing, it appears that the greater the deviation of the coefficient from a value of about 0.0000083, the greater the surface disintegration. No definite relationship is indicated between coefficient of expansion and loss of strength due to freezing and thawing.

Results indicate that relative durability depends upon some interrelationship between some properties of both cement and aggregate. In trying to determine the cause of lack of durability of concrete, it may not be safe to fix the blame upon one of the materials by a process of elimination from observations of the behavior of each of the materials in other combinations.

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Tests on Concrete Masonry Units Using Tamping and Vibration Molding Methods*

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SYNOPSIS

This paper presents results of a comprehensive, correlated series of tests on concrete masonry units made with seven different aggregates, cinders, Haydite, limestone, Pottsco, sand and gravel, Superock and Waylite. Comparative data show the effect of two different types of molding, vibration and tamping, on compressive strength, absorption, capillarity, specific weight, durability, volume change and thermal expansion coefficient for each aggregate. Similar comparative data are presented for variations in cement content for each aggregate.

The increasing use of concrete masonry units in all types of construction and the recent introduction of new aggregates and vibratory methods of molding in the manufacture of such units have presented several problems for which there were no definite general answers. An attack on the more pressing of these was made possible through the sponsorship of a test program by the National Concrete Masonry Association in cooperation with the University of Wisconsin and the following aggregate and equipment producers: The American Aggregate Co., Kansas City, Mo., The Besser Manufacturing Co., Alpena, Mich., The Celotex Corp., Chicago, Ill., The National Cinder Concrete Products Association, Philadelphia, Penn., The Stearns Manufacturing Co., Adrian, Mich., The Superock Co., Birmingham, Ala., and The Waylite Co., Chicago, Ill.

The present program was designed to answer, at least in part, questions as to the relative effects of tamping versus vibration on the

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compressive strength, durability, yield, absorption, specific weight and volume constancy of units from identical mixes; the effect of yield in terms of units per sack of cement on weight, absorption, durability, strength, and volume constancy; and the relationship of durability to strength and absorption.

Seven aggregates were included: cinders, Haydite, limestone, Pottsco, sand and gravel, Superock, and Waylite. It was necessary to limit these tests to one grading of one representative aggregate for each type and to limit molding practice to one method of procedure on one make of tamping and one make of vibration equipment.

The physical tests included determinations of weight per cubic foot and sieve analyses on fine, coarse and combined aggregates; compressive strength at 7 and 28 days, absorption at 28 days, weight per cu. ft. of concrete, capillarity and yield on 8 x 8 x 16-in. 3 cell, 45 per cent core area units; compressive strength at 170 days, volume change through two cycles of wetting and drying, temperature coefficient of expansion and resistance to 100 cycles of freezing and thawing on 8 x 8 x 8-in. halves of full sized units.

MATERIALS

All the materials used in the manufacture of the concrete masonry units were selected and handled in accordance with standard plant practice in so far as it was compatible with test conditions.

Portland Cement

The portland cement used in the manufacture of all the units was a blend of equal parts by weight of four different brands. Two of these were regularly used at the Franklin Park plant and the other two at the Milwaukee plant where the test units were manufactured. Forty sacks of each brand were selected at random, twenty of which were shipped by truck to the other cooperating plant.

Aggregates

Eight cubic yards of each of the seven different aggregates used in this investigation were obtained from stocks of aggregate, in excess of 200 cu. yd., at products plants regularly using that particular type of aggregate. In general the aggregates were obtained from plants which were closest to one of the two plants where the test units were to be made and cured.

The Waylite and Pottsco aggregates were obtained from plants at Franklin Park, Ill., the limestone from a plant at Elmhurst, Ill.; Haydite and sand and gravel aggregates from a plant at Milwaukee, Wis.; the cinders from a Detroit, Mich., plant and the Superock was obtained from a plant at Nashville, Tenn.

At each of the plants Mr. Woodworth superintended the selection and sacking of the aggregates. Approximately 8 cu. yd. of each aggregate, 200 sacks, were placed in cleaned cement sacks. To facilitate batching at the time of manufacture equal weights were placed in each sack for a particular type or grading of aggregate. The sacked aggregate was shipped by truck to one of the cooperating plants and then divided equally into two parts, one part being shipped by truck to the other cooperating plant.

The materials available at each of the two plants when finally assembled consisted of 100 sacks each of seven different aggregates and 20 sacks each of the 4 different brands of cement.

Water used in mixing was that available at each of the plants and was the only variable in type of material.

MANUFACTURE

Proportioning

The following general grading limitations were set up as a guide: coarse aggregate to pass a ½-in. sieve, fine aggregate to pass a No. 4 sieve, combined aggregate to be proportioned so that not less than 40 per cent nor more than 60 per cent of fine aggregate shall be used. Proportions within these limits were determined by the aggregate producers for the Haydite, Waylite, Superock and Pottsco mixes and by Mr. Woodworth for the limestone, cinder, and sand and gravel mixes.

Approximate yield in terms of units per sack and water content of batch were determined in trial batches at each of the plants prior to manufacture of test units. Due to different methods of compaction it was found necessary in some cases to change the total amount of aggregate used to provide equivalent yields for both types of molding equipment.

Mixing Procedure

Batches designed to produce 45 block, (1¼ in. face shell, 45 per cent core area) with each type of molding equipment were mixed in following manner.

For the five lightweight aggregates, all of the aggregate and from ½ to ⅔ of the mixing water were mixed for 3 minutes in spiral blade mixers of 28 cu. ft. capacity. The cement was then added with the balance of the mixing water and mixing continued for 5 more minutes. For the two heavyweight aggregates all of the materials were added and then mixed for 5 minutes.

Molding Equipment

Vibration molding equipment was that installed in the plant of the Midwest Concrete Pipe Co. and consisted of a Stearns Joltcrete

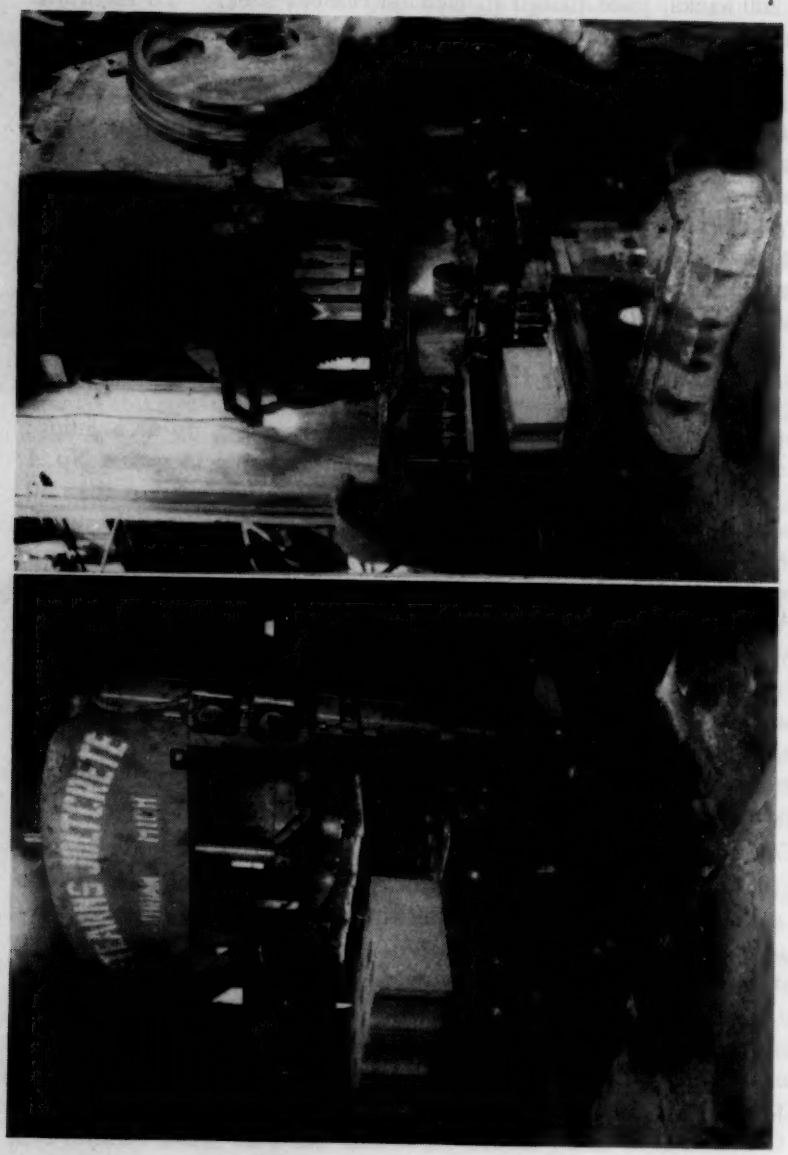


FIG. 1 AND 2—STEARNS JOLTCRETE VIBRATION EQUIPMENT AND BESSER HEAVY DUTY TAMPING EQUIPMENT USED IN MANUFACTURE OF UNITS

machine equipped to produce 2, 8 x 8 x 16-in. 3 oval-core units and 2, 8 x 8 x 8-in. halves of 3 oval-core units in a cycle. Tamping molding equipment was that installed in the plant of the Best Block Co., Milwaukee, Wis., and consisted of two Besser Heavy Duty Power Strippers, one equipped to make an 8 x 8 x 16-in. 3 rectangular core unit and the other to produce two half-units. In general about 30 half units and 30 full units were made from each batch. The regular plant crews operated the machines under the direction of a representative of the manufacturer of the equipment.

Yield

Average weight per unit was determined by weighing 10 full units immediately after molding. Yield was calculated by dividing the total weight of all materials added at the mixer by the average weight to determine the number of units per batch, from which the number of units per sack of cement was obtained.

Curing

Units were cured for the first 24 hours by placing them in the regular moist curing rooms in use at each plant at a temperature of between 90°–110° F. and a saturated atmosphere produced by steam jets opening directly into the room.

Subsequent to curing the units were stockpiled in the usual manner, under cover, until shipment to the laboratories for tests.

Units for compression tests at 7 days, volume change and temperature coefficient determinations were shipped to the Research Laboratory of the Portland Cement Association four days after manufacture.

Units for compression tests at 28 days and at 6 months, and for absorption, capillarity and durability tests were shipped to the Materials Testing Laboratory of the University of Wisconsin eighteen days after manufacture.

TESTING PROCEDURE

The compression tests, absorption, weight of concrete per cubic foot, and void determinations were made in accordance with A. S. T. M. Specification, Methods of Sampling and Testing Concrete Masonry Units, C140-38T, and Federal Specification SS-C-621.

Freezing and Thawing Tests

Durability was determined by subjecting four half units for each variable to 100 cycles of freezing and thawing. At an age of 60 days the units were completely immersed in water for 48 hours, weighed after draining for one minute, and then placed face down with cells horizontal in trays in which the water level was maintained at a depth of ¾-in. The loaded trays were placed in a freezer in which the air

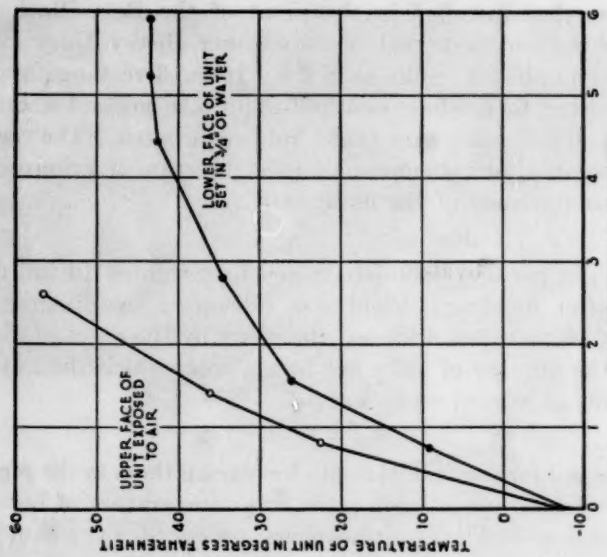
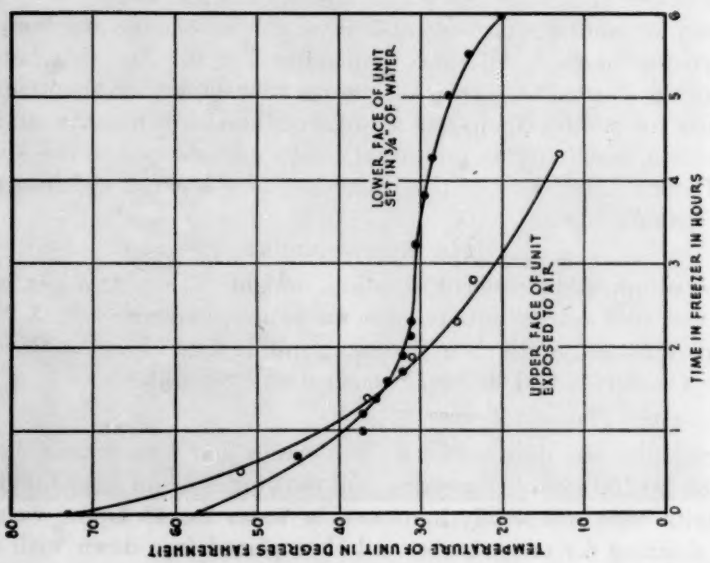


FIG. 3 (LEFT) AND 4—TIME-TEMPERATURE RELATIONS FOR FREEZING AND THAWING CYCLES USED



temperature ranged from 0° to -10° F. and left until the block temperatures had been reduced to the same range. The average temperature at the end of the freezing cycle was -8° F. and the average time required to reduce the block temperature from 32° F. to 28° F. was ½ hour for the top face and 2½ hours for the face immersed in water.

After removal from the freezer the units were allowed to thaw in laboratory air. The thawing rate was high due to the constant circulation of warm air from unit heaters located near the ceiling of the thawing room. Typical temperature-time curves for the freezing and thawing cycles are shown in Fig. 3 and 4.

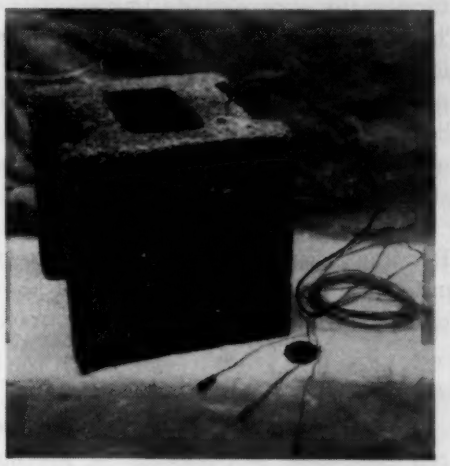


FIG. 4(A)—

One complete cycle of freezing and thawing was secured each weekday except when inspections of units were made. Units were examined at intervals throughout the program for visible damage, and weight changes were determined at the end of 50 and 100 cycles by again weighing the units after 48 hours' immersion in water.

At the conclusion of the 100 cycle program the units were allowed to dry for two weeks in air and were then broken in compression. At the same time control specimens, which were companion halves of units in the freezing and thawing program and had been stored for the entire period in the thawing room, were also tested.

Rate of Absorption and Capillarity

Units used for the 28-day absorption tests were dried and absorptions after complete immersion in water for various times from ½ to 24 hours were determined. After redrying, the same units were placed

on racks with cells vertical and set in water to a depth of ¼-in. The suction rate, expressed in pounds of water absorbed per block was determined by frequent weighings during a 24-hour period, and the average height of water in the block at the end of the period was noted.

Volume Change Tests

Volume change due to wetting and drying was determined by subjecting three half units for each variable to two cycles of wetting and drying. Sixty-day-old room dry units with brass strain gage plugs 8 in. on center, set diagonally on one face of the unit, were completely immersed in water at 70°-75° F. for 48 hours. Strain gage readings and weight determinations were made immediately before and after immersion. The units were then stacked in single rows on ¾-in. square separation sticks to facilitate uniform drying. Gage readings and weight determinations were made at 2, 5, 12, 19 and 28 days after immersion. The second cycle was repeated in the same manner.

Volume change due to temperature changes was determined by strain gage readings on one of the half units for each variable, which had previously been used in the volume change studies for wetting and drying. Readings were made over a range of approximately 130° F. The units were held at temperatures of about 125° F. for 24 hours, 80° F. for 18 hours, and -7° F. for 16 hours before taking the readings.

Acknowledgments

The authors wish to acknowledge the valuable assistance of Paul Klieger, Research Assistant, Department of Mechanics, University of Wisconsin, who was responsible for the detail testing conducted at the University, and Sherman Bryan, Assistant Engineer, Research Laboratory of the Portland Cement Association, Chicago, who conducted the volume change and thermal coefficient studies.

The authors also wish to make special mention of the excellent cooperation accorded them during the manufacture of the units for the test program by the entire personnel of the Best Block Co., Milwaukee, and the Mid-West Concrete Pipe Co., Franklin Park. Ill.

DISCUSSION

The principal data on aggregate properties, mix, strength, absorption, durability, and volume change, together with brief comments are presented individually for each material with summary tables and curves for certain variables for which at least partial relationships could be established. Tabular values and plotted points represent tests on 4 or 5 units, except for temperature coefficient and moisture-volume change where 1 and 3 units respectively were used.

Yields in all cases closely approached the desired 17, 25, and 33 units per sack for which the mixes were designed. Mix proportions and water contents, however, were varied for different classes of materials. Mixing water is based on water actually added at the mixer plus moisture held in the aggregate, with no corrections for absorption.

Absorptions for all mixes in the program were below the permissible limits of Federal and A. S. T. M. Specifications for Load Bearing Units, but strengths in several cases fell below the 700 p.s.i. requirement. The latter units were not included in the original durability program but were later run for 50 cycles as freezer space became available, to complete the data needed to establish fair durability ratings and equitable specification requirements.

Durability ratings were arbitrarily based on 3 factors: weight loss and the ratios of strength after 100 cycles of freezing and thawing to the strengths determined at 28 and at 170 days. To merit a rating of "excellent," no loss of weight was permitted and units were required to show a minimum strength ratio of either 0.95 at 28 days or 0.90 at 170 days. A rating of "good" was used when the weight loss did not exceed 2 per cent and strength ratios were over 0.90 or 0.85 respectively. In several instances a rating of "fair" was used where disintegration was confined to a single specimen, the average weight loss was less than 15 per cent, and strength ratios were within the limits prescribed for a "good" rating.

Cinder Units

All of the cinder units passed requirements of the Federal and A. S. T. M. Specifications for Load Bearing Masonry Units, but only the 17-unit-per-sack mixes behaved sufficiently well under freezing and thawing to merit ratings above "failure." Curves relating method of molding and cement content to compressive strength, absorption and specific weight (Fig. 5) showed normal trends. Vibrated mixes were lighter, absorbed less water and were stronger than tamped mixes, the latter difference being most pronounced at an age of 7 days and after 6 months laboratory storage. (Table 2). The two richer tamped mixes, however, withstood freezing and thawing better than did the corresponding vibrated mixes. (Table 3).

At the end of 50 cycles the 25- and 33-unit-per-sack vibrated mixes and the leanest tamped mix showed losses in weight from 36 to 47 per cent (Fig. 6 and 7). The ratios of strength after completion of 100 cycles of freezing and thawing to the strength of the control specimens for the balance of the mixes were 0.55 for T2, 0.84 for V1,

TABLE 1—PHYSICAL DATA ON CINDER AGGREGATE

Grading	Weight, Lb. per Cu. Ft.		Moisture Content as Sampled	Sieve Analysis % Coarser Than							Fineness Modulus
	Damp Loose	Dry Rodded		⅜	4	8	16	30	50	100	
Fine	59.0	70.0	4.7			4	37	69	81	89	2.80
Coarse	66.2	74.5	4.0	3	27	44	72	82	88	93	4.10
Combined Fine 20% Coarse 80%		76.0		3	21	36	65	79	87	92	3.83

TABLE 2—BATCH DETAILS AND PHYSICAL DATA ON CINDER UNITS

Mix No.	Batch Details					Physical Measurements of Units										
	Proportions per Sack of Cement					Weight of		Compressive Strength*					Absorption		Capillarity**		
	By	Aggregate		Mixing Water, Gal.	Yield, No. of Units	Unit at 28 Da., Lb.	Concrete, Lb./Ft.³	Lb. per Sq. In. Gross Area				Ratio, 100 Cy. 28 Da.	Lb./Ft.³ in 24 Hr.	Ratio, ½ Hr. 24 Hr.	Absorption, Lb. per Unit		Water Height in. 24 Hr.
		Fine	Coarse					7 Da.	28 Da.	170 Da.	100 F&T Cycles				½ Hr.	24 Hr.	
V1	Wt.	89	376	5.4	18	32.0	98	1530	1680	1880	1590	.94	10.0	0.86	0.15	0.40	1½
	Vol.	1.2	4.8														
V2	Wt.	132	560	7.8	27	30.6	96	950	1040	1120	0	0	12.1	0.88	0.45	0.65	2¼
	Vol.	1.8	7.2														
V3	Wt.	175	740	10.2	35	30.0	93	690	730	800	0	0	12.1	0.88	0.50	1.00	3
	Vol.	2.4	9.6														
T1	Wt.	89	376	5.2	17.5	32.9	104	1280	1610	1540	1880	1.16	11.3	0.72	0.30	0.80	2½
	Vol.	1.2	4.8					(0.84)*	(0.96)	(0.82)	(1.18)						
T2	Wt.	132	560	7.8	26	31.1	99	810	970	1160	640	0.66	12.2	0.90	0.45	1.05	3
	Vol.	1.8	7.2					(0.85)	(0.93)	(1.04)	(tamped)						
T3	Wt.	175	740	10.2	35	29.7	96	510	700	590	0	0	12.9	0.92	0.80	1.40	4
	Vol.	2.4	9.6					(0.74)	(0.96)	(0.74)							

*Figures in parentheses show strength ratios of tamped to vibrated units at a given age.
**Units set face down in ¼ inch of water.

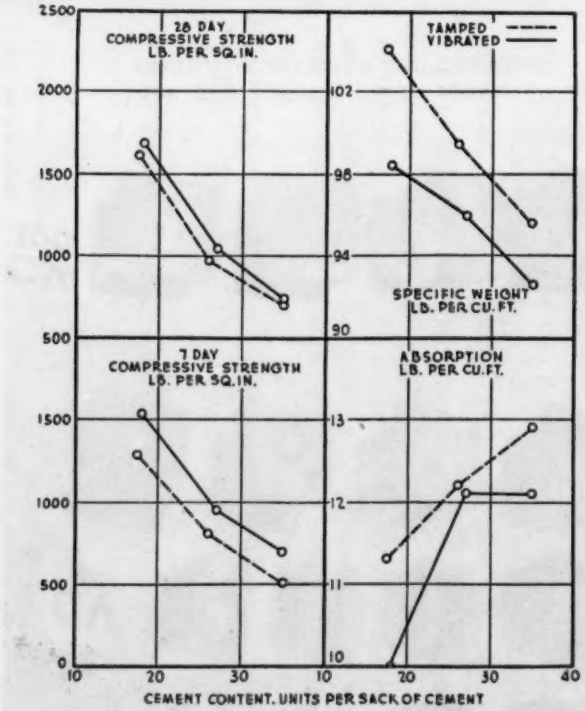


FIG. 5—EFFECT OF METHOD OF MOLDING AND CEMENT CONTENT ON COMPRESSIVE STRENGTH, ABSORPTION, AND SPECIFIC WEIGHT OF CINDER UNITS

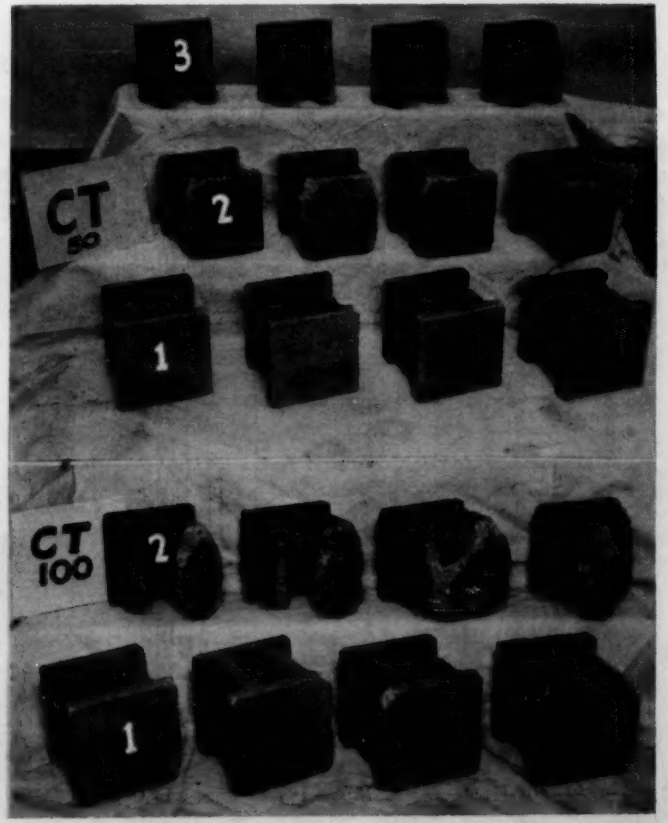


FIG. 6 (RIGHT)—TAMPED CINDER UNITS AFTER 50 AND 100 CYCLES OF FREEZING AND THAWING

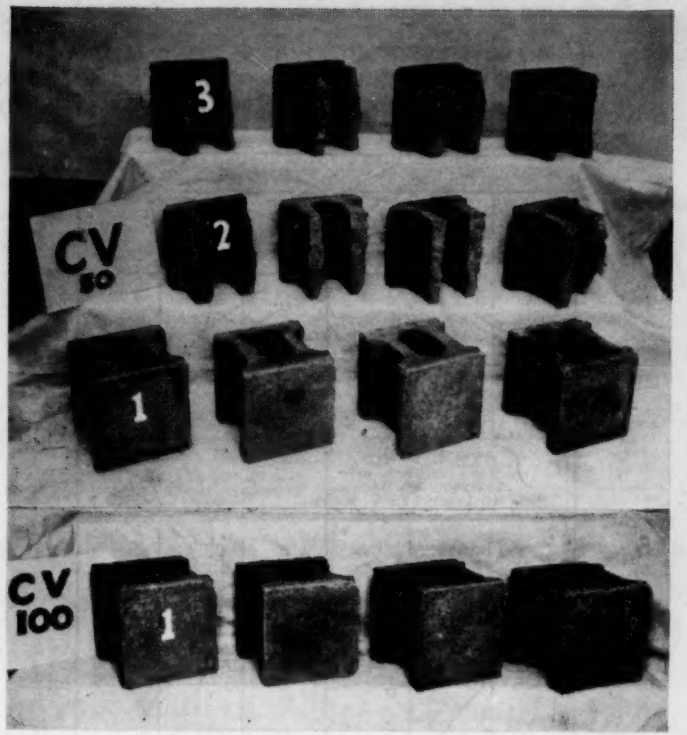


FIG. 7 — VIBRATED CINDER UNITS AFTER 50 AND 100 CYCLES OF FREEZING AND THAWING

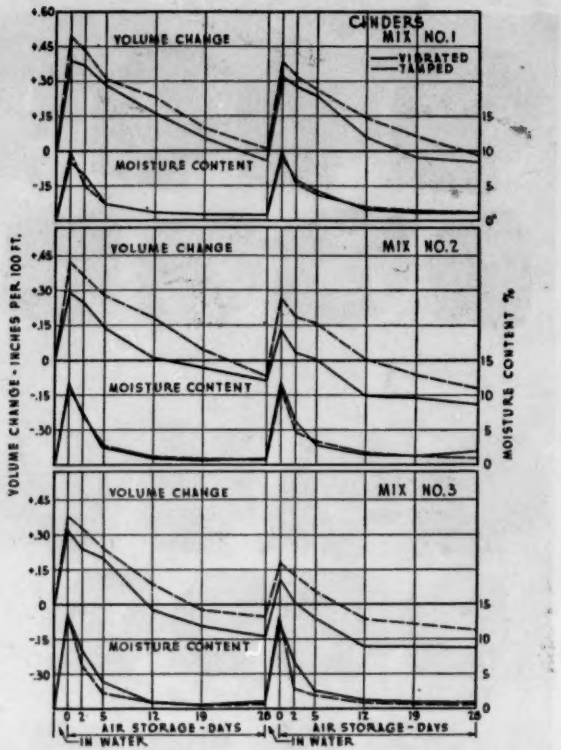


FIG. 8—VOLUME CHANGE DUE TO CHANGES IN MOISTURE CONTENT OF UNIT

TABLE 3—DURABILITY DATA FOR CINDER UNITS

Mix No.	Percentage Change in Weight at Cycle Number		Summary of Observations on Freezing and Thawing Specimens At Cycle Number						Rating
	50	100	15	30	50	65	80	100	
V1	+ 1.6	+ 1.6			x	xx	xx	xx	Good
V2	−36.4	—	x	*	36%	—	Removed	—	Failure
V3	−38.6	—	x	***	39%	—	Removed	—	Failure
T1	+ 0.6	+ 0.1			x	xxx	xxx	*	Excellent
T2	− 3.7	−24.5		*	**	***	***	***	Failure
T3	−47.1	—	x	**	47%	—	Removed	—	Failure

x Very light spalling
xx Light spalling
xxx Light to medium spalling
* Medium spalling
** Heavy spalling
*** Disintegration

and 1.22 for mix T1. Durability ratings, therefore, are "excellent" for mix T1, "good" for mix V1, and "failure" for all others.

The average thermal coefficient of expansion per degree Fahrenheit for the vibrated units was .0000021 and for the tamped units .0000022.

As expected, the rate of absorption and suction rate decreased as the richness of mix increased (Table 2). It is interesting to note that by far the lowest rate of absorption, 0.72, occurred for the one mix which rated "excellent" under freezing and thawing conditions.

Haydite Units

The Haydite units were all well above the minimum requirements of Federal and A. S. T. M. Specifications for Load Bearing Masonry Units, the lowest 28-day compressive strength being 970 p.s.i. for mix T3. The curves of Fig. 9 show that vibrated units are somewhat lighter than tamped units but are considerably stronger and absorb less water. These effects are most pronounced for the richest mix.

All of the Haydite mixes withstood 100 cycles of freezing and thawing without visible damage (Fig. 10 and 11); and the strength after such treatment was in all instances greater than the strength at 28-days (Table 5). The superiority in strength of the vibrated mixes was maintained during the freezing and thawing treatment for the two richer mixes, but for the 33-unit-per-sack mixes the tamped units showed a remarkable gain in strength and surpassed those of the vibrated mixes. The durability rating for all mixes is "excellent." (Table 6).

The average thermal coefficient of expansion per degree Fahrenheit for the vibrated units was .0000039 and for the tamped units .0000034.

For both methods of placement the rate of absorption and suction rate values are lowest for the richest mixes (Table 5), but values for the two leaner mixes are almost identical.

TABLE 4—PHYSICAL DATA ON HAYDITE AGGREGATE

Grading	Weight, Lb. per Cu. Ft.		Moisture Content as Sampled	Sieve Analysis % Coarser Than							Fineness Modulus
	Damp Loose	Dry Rodded		⅜	4	8	16	30	50	100	
Fine	51.9	62.2	6			13	43	61	69	78	2.64
Coarse	45.7	46.0	6	5	70	96	97	97	97	98	5.60
Combined Fine 45% Coarse 55%		58.5		2	29	40	66	76	81	87	3.81

TABLE 5—BATCH DETAILS AND PHYSICAL DATA ON HAYDITE UNITS

Mix No.	Batch Details					Physical Measurements of Units										
	Proportions per Sack of Cement					Weight of		Compressive Strength*					Absorption		Capillarity**		
	By	Aggregate		Mixing Water, Gal.	Yield, No. of Units	Unit at 28 Da., Lb.	Concrete, Lb./Ft.³	Lb. per Sq. In. Gross Area				Ratio, 100 Cy. 28 Da.	Lb./Ft.³ in 24 Hr.	Ratio, ½ Hr. 24 Hr.	Absorption, Lb. per Unit		Water Height in. 24 Hr.
		Fine	Coarse					7 Da.	28 Da.	170 Da.	100 F&T Cycles				½ Hr.	24 Hr.	
V1	Wt.	172	157	6.2	16.8	26.7	83	1640	1810	2080	2100	1.16	7.9	0.59	0.20	0.45	1¼
	Vol.	2.55	3.25														
V2	Wt.	255	232	7.8	25.8	24.3	78	1190	1280	1520	1480	1.16	13.1	0.87	0.50	0.80	2
	Vol.	3.80	4.80														
V3	Wt.	339	308	11.0	32.8	24.6	76	1000	1150	1410	1250	1.09	13.7	0.84	0.45	0.80	2
	Vol.	5.00	6.35														
T1	Wt.	172	157	6.3	16.4	27.3	84	1250	1450	1540	1980	1.36	12.1	0.64	0.40	0.55	1½
	Vol.	2.55	3.25					(0.76)*	(0.80)	(0.74)	(0.95)						
T2	Wt.	255	232	8.9	24.4	25.3	80	950	1170	1090	1350	1.16	13.6	0.72	0.50	0.80	2
	Vol.	3.80	4.80					(0.80)	(0.91)	(0.72)	(0.91)						
T3	Wt.	339	308	11.0	31.8	23.7	77	690	970	1000	1340	1.39	14.2	0.76	0.50	0.95	3
	Vol.	5.00	6.35					(0.69)	(0.84)	(0.71)	(1.08)						

*Figures in parentheses show strength ratios of tamped to vibrated units at a given age.
**Units set face down in ¼ inch of water.

TABLE 6—DURABILITY DATA FOR HAYDITE UNITS

Mix No.	Percentage Change in Weight at Cycle Number		Summary of Observations on Freezing and Thawing Specimens At Cycle Number						Rating
	50	100	15	30	50	65	80	100	
V1	+1.1	+1.0				x	x	x	Excellent
V2	+1.8	+2.1		x	xx	xx	xx	xx	Excellent
V3	+2.2	+1.1			x	xx	xx	xx	Excellent
T1	+1.7	+1.6			x	x	xx	xx	Excellent
T2	+1.3	+1.2		x	xx	xx	xxx	xxx	Excellent
T3	+2.2	+1.9	x	xx	xxx	xxx	xxx	xxx	Excellent

x Very light spalling
xx Light spalling
xxx Light to medium spalling

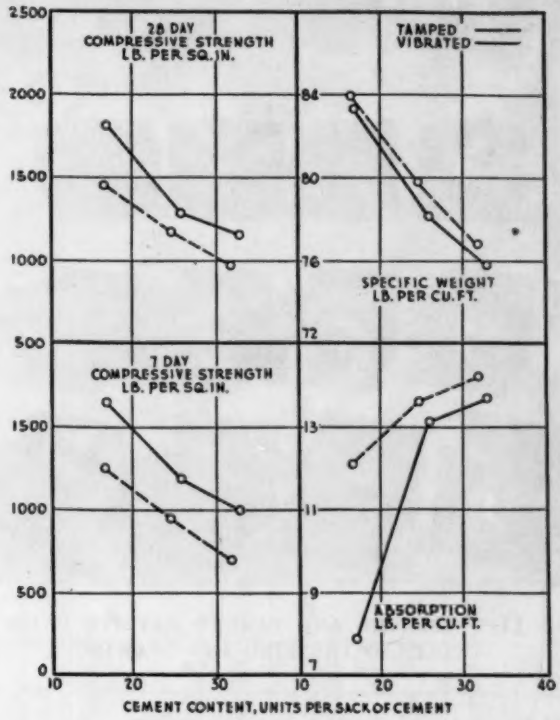


FIG. 9—EFFECT OF METHOD OF MOLDING AND CEMENT CONTENT ON COMPRESSIVE STRENGTH, ABSORPTION, AND SPECIFIC WEIGHT OF HAYDITE UNITS

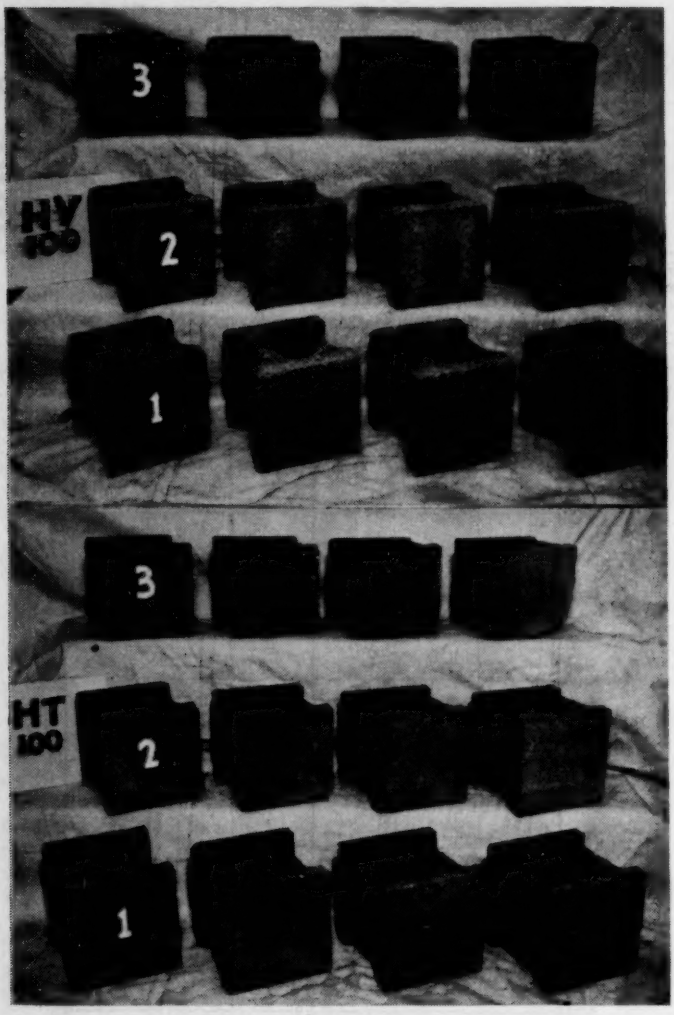


FIG. 10 AND 11—VIBRATED AND TAMPED HAYDITE UNITS AFTER 100 CYCLES OF FREEZING AND THAWING

Limestone Units

All of the limestone units passed the Federal and A. S. T. M. Specifications for Load Bearing Masonry Units, but only the richest of the tamped mixes was reasonably satisfactory from the durability standpoint (Table 8). Disintegration of the leaner mixes was pronounced even before the regular 15 cycle inspection. In the leaner

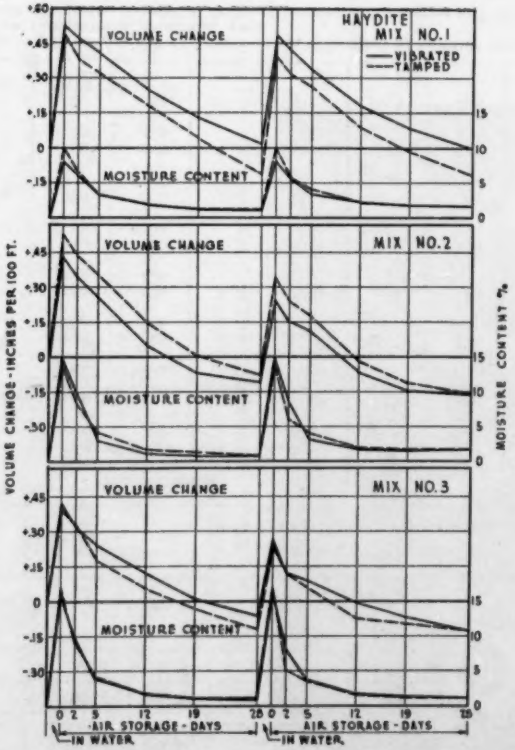


FIG. 12—VOLUME CHANGE DUE TO CHANGES IN MOISTURE CONTENT OF UNIT

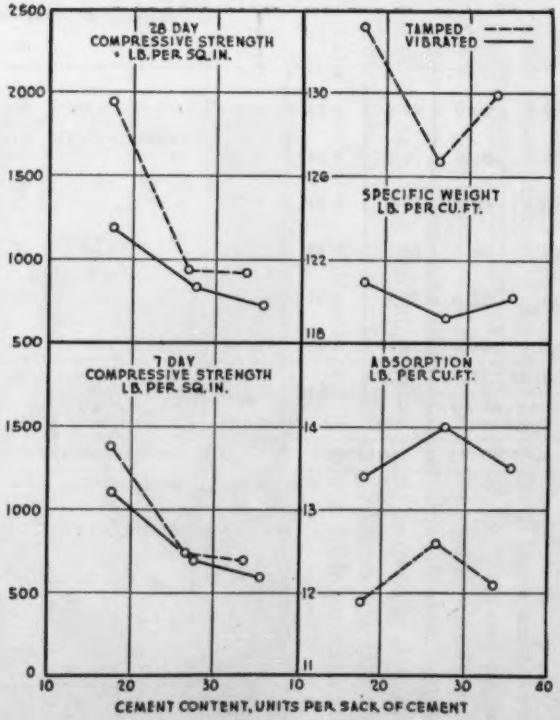

28 DAY COMPRESSIVE STRENGTH - LB. PER. SQ. IN.
TAMPED
VIBRATED
SPECIFIC WEIGHT LB. PER CU. FT.
7 DAY COMPRESSIVE STRENGTH LB. PER. SQ. IN.
ABSORPTION LB. PER CU. FT.
CEMENT CONTENT, UNITS PER SACK OF CEMENT

FIG. 13—EFFECT OF METHOD OF MOLDING AND CEMENT CONTENT ON COMPRESSIVE STRENGTH, ABSORPTION, AND SPECIFIC WEIGHT OF LIMESTONE UNITS

TABLE 7—PHYSICAL DATA ON LIMESTONE AGGREGATE

Grading	Weight, Lb. per Cu. Ft.		Moisture Content as Sampled	Sieve Analysis % Coarser Than							Fineness Modulus
	Damp Loose	Dry Rodded		⅜	4	8	16	30	50	100	
Fine	91.4	100.5	4.7		5	31	56	73	83	89	3.37
Coarse	85.2	92.0	1.4		46	92	96	97	97	98	5.27
Combined Fine 50% Coarse 50%		105.0			24	58	74	84	90	93	4.23

TABLE 8—BATCH DETAILS AND PHYSICAL DATA ON LIMESTONE UNITS

Mix No.	Batch Details					Physical Measurements of Units										
		Proportions per Sack of Cement				Weight of		Compressive Strength*					Absorption		Capillarity**		
		Aggregate		Mixing Water, Gal.	Yield, No. of Units	Unit at 28 Da., Lb.	Concrete, Lb./Ft.³	Lb. per Sq. In. Gross Area				Ratio, 100 Cy. 28 Da.	Lb./Ft.³ in 24 Hr.	Ratio, ½ Hr. 24 Hr.	Absorption, Lb. per Unit		Water Height in. 24 Hr.
	By	Fine	Coarse					7 Da.	28 Da.	170 Da.	100 F&T Cycles				½ Hr.	24 Hr.	
V1	Wt.	320	288	4.3	17.8	36.9	121	1110	1180	1560	700	0.59	13.4	0.91	0.75	1.05	3
	Vol.	3.0	2.1														
V2	Wt.	475	430	6.1	27.4	35.7	119	690	830	870[a]	0	0	14.0	0.92	0.75	1.15	3
	Vol.	4.5	4.6														
V3	Wt.	630	570	7.0	35.4	35.8	120	590	720	740[a]	0	0	13.5	0.92	0.85	1.20	3½
	Vol.	5.95	6.05														
T1	Wt.	336	300	5.2	17.5	43.3	133	1380	1940	2370	2190	1.13	11.9	0.64	0.35	1.05	2½
	Vol.	3.2	3.25					(1.24)	(1.64)	(1.51)	(3.15)						
T2	Wt.	500	450	7.8	26.4	39.4	127	740	930	1280	210	0.23	12.6	0.91	0.70	1.35	3
	Vol.	4.75	4.8					(1.07)	(1.12)	(1.47)[a]	(tamped)						
T3	Wt.	660	595	10.2	33.3	40.9	130	690	910	970	0	0	12.1	0.88	0.50	1.40	3
	Vol.	6.3	6.4					(1.17)	(1.27)	(1.31)[a]							

*Figures in parentheses show strength ratios of tamped to vibrated units at a given age.
**Units set face down in ¼ inch of water.
[a]Specimens subjected to only 50 cycles of freezing and thawing and control specimens tested at 270 days.

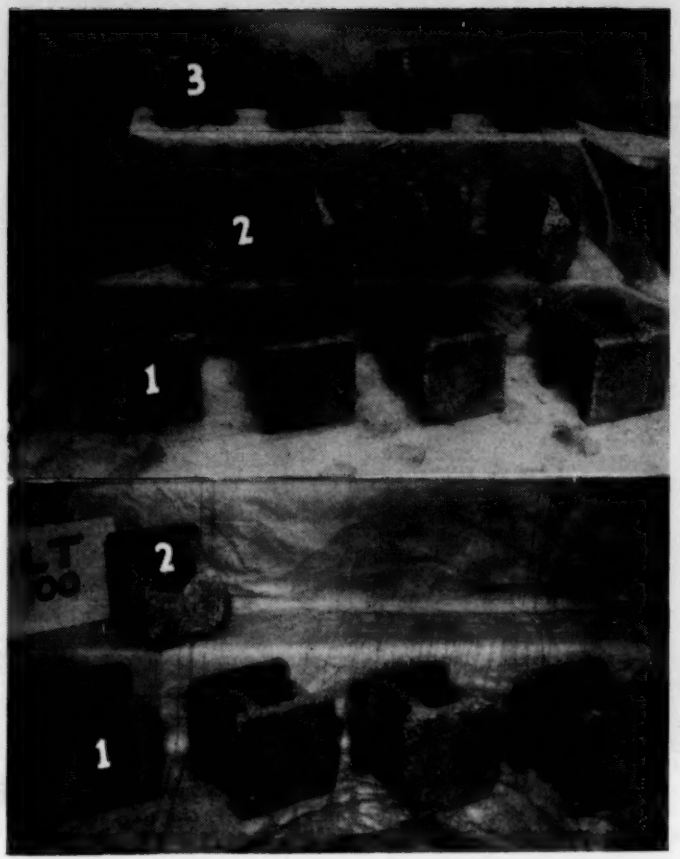
mixes bond failures occurred very early, whereas in the richer mixes the aggregate began to pop on the surface of the units. This is well illustrated in Fig. 14 and 15. Mix T1 is rated "good" but all other mixes "failure." The poor behavior of these units under freezing and thawing conditions, as evidenced by popping and crumbling, was probably due in part to unsoundness of the particular limestone aggregate. Another contributing factor for the vibrated limestone mixes was the fact that the mixes were made extremely dry to overcome difficulties in placing due to excessive stickiness of mixes with normal water contents. This resulted in units with an extremely coarse

FIG. 14—VIBRATED LIMESTONE UNITS AFTER 50 AND 100 CYCLES OF FREEZING AND THAWING

texture and a high percentage of voids which is reflected in the weights of the units for all three mixes.

The average thermal coefficient of expansion per degree Fahrenheit for the vibrated units was .0000051 and for the tamped units .0000050.

Rate of absorption and suction rate values (Table 8) are almost identical for the 3 vibrated mixes and the two leaner tamped mixes. Again it is interesting to note that the value for the one good mix is much below the others.



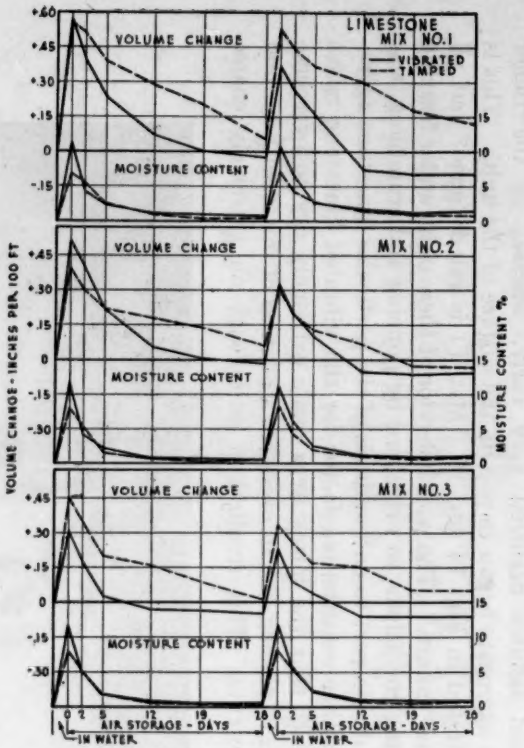


FIG. 15 (LEFT)—TAMPED LIMESTONE UNITS AFTER 50 AND 100 CYCLES OF FREEZING AND THAWING

FIG. 16—VOLUME CHANGE DUE TO CHANGES IN MOISTURE CONTENT OF UNIT

TABLE 9—DURABILITY DATA FOR LIMESTONE UNITS

Mix No.	Percentage Change in Weight at Cycle Number		Summary of Observations on Freezing and Thawing Specimens At Cycle Number						Rating
	50	100	15	30	50	65	80	100	
V1	− 2.9	−20.1	xx	**	**	***	***	***	Failure
V2	−66.6	50 only	***	***	***	End of test 50 cycles			Failure
V3	−82.9	50 only	***	***	***	End of test 50 cycles			Failure
T1	− 0.1	− 0.7		x	*	*	**	**	Good
T2	−30.1	−81.0	xx	**	30%	3 of 4 specimens removed			Failure
T3	−60.6	—	***	***	60%	all specimens removed			Failure

x Very light spalling
xx Light spalling
xxx Light to medium spalling
* Medium spalling
** Heavy spalling
*** Disintegration

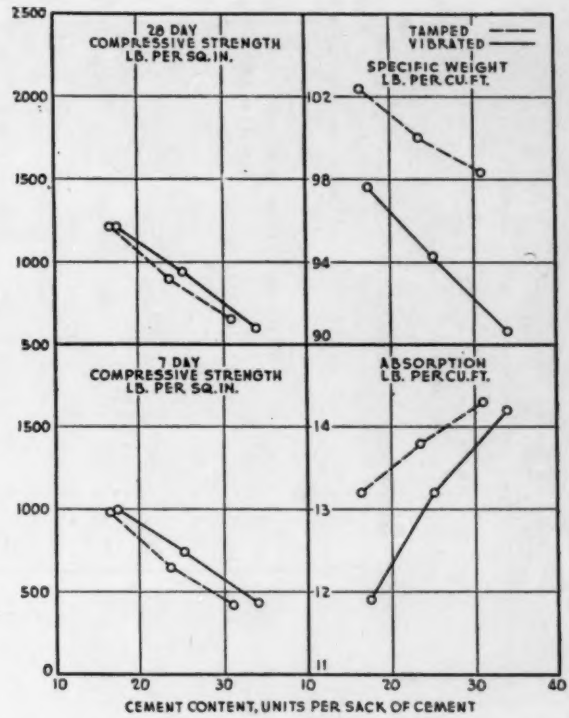


FIG. 17—EFFECT OF METHOD OF MOLDING AND CEMENT CONTENT ON COMPRESSIVE STRENGTH, ABSORPTION, AND SPECIFIC WEIGHT OF POTTSCO UNITS

Pottsco Units

Only the 17- and 25-unit-per-sack mixes passed the strength requirements of the Federal and A. S. T. M. Specifications for Load Bearing Masonry Units (Table 11). The effects of method of molding and cement content on compressive strength, absorption, and specific

TABLE 10—PHYSICAL DATA ON POTTSCO AGGREGATE

Grading	Weight, Lb. per Cu. Ft.		Moisture Content as Sampled	Sieve Analysis % Coarser Than							Fineness Modulus
	Damp Loose	Dry Rodded		¾	4	8	16	30	50	100	
Fine	57.4	64.5	7.5		3	24	45	61	75	84	2.92
Coarse	55.2	54.5	7.0	18	73	85	87	87	90	93	5.33
Combined Fine 50% Coarse 50%		66.0		8	35	52	64	73	82	88	4.02

TABLE 11—BATCH DETAILS AND PHYSICAL DATA ON POTTSCO UNITS

Mix No.	Batch Details					Physical Measurements of Units										
		Proportions per Sack of Cement				Weight of		Compressive Strength*					Absorption		Capillarity**		
								Lb. per Sq. In. Gross Area				Ratio,	Lb./Ft.³ in	Ratio,	Absorption, Lb. per Unit		Water Height in.
	By	Aggregate		Mixing Water, Gal.	Yield, No. of Units	Unit at 28 Da., Lb.	Concrete, Lb./Ft.³	7 Da.	28 Da.	170 Da.	100 F&T Cycles	100 Cy. / 28 Da.	24 Hr.	½ Hr. / 24 Hr.	½ Hr.	24 Hr.	24 Hr.
		Fine	Coarse														
V1	Wt.	228	193	5.9	17.3	30.6	98	990	1210	1580	1440	1.20	11.9	0.88	0.30	0.60	1½
	Vol.	3.3	3.3														
V2	Wt.	340	287	8.8	25.1	30.5	94	740	940	1220	480	0.52	13.2	0.89	0.25	0.70	2
	Vol.	4.9	4.9														
V3	Wt.	447	378	11.6	34.0	27.8	91	430	600	650[a]	270[a]	0.45[a]	14.2	0.89	0.45	1.00	3
	Vol.	6.5	6.5														
T1	Wt.	228	193	6.0	16.4	33.0	102	980	1210	1320	1620	1.34	13.2	0.67	0.30	0.70	2½
	Vol.	3.3	3.3					(0.99)	(1.00)	(0.83)	(1.12)						
T2	Wt.	340	287	8.9	23.6	32.1	100	650	890	1090	1190	1.34	13.8	0.74	0.30	0.95	2½
	Vol.	4.9	4.9					(0.88)	(0.95)	(0.90)	(2.45)						
T3	Wt.	447	378	11.7	31.0	31.4	98	420	660	880[a]	610[a]	0.92[a]	14.3	0.84	0.50	1.65	4
	Vol.	6.5	6.5					(0.98)	(1.10)	(1.35)	(2.26)[a]						

*Figures in parentheses show strength ratios of tamped to vibrated units at a given age.
**Units set face down in ¼ inch of water.
[a]Specimens subjected to only 50 cycles of freezing and thawing and control specimens tested at 270 days.

TABLE 12—DURABILITY DATA FOR POTTSCO UNITS

Mix No.	Percentage Change in Weight at Cycle Number		Summary of Observations on Freezing and Thawing Specimens At Cycle Number						Rating
	50	100	15	30	50	65	80	100	
V1	+ 1.6	− 1.3					x	x	Excellent
V2	− 4.7	−47.3		x	**	***	***	***	Failure
V3	−37.7	50 only	xx	**	***	End of test 50 cycles			Failure
T1	+ 1.2	+ 1.1			x	xx	xx	xx	Excellent
T2	− 0.1	− 7.8		x	xxx	**	***	***	Fair
T3	−19.1	50 only	xx	*	***	End of test 50 cycles			Failure

x Very light spalling
xx Light spalling
xxx Light to medium spalling
* Medium spalling
** Heavy spalling
*** Disintegration

FIG. 18—VIBRATED POTTSCO UNITS AFTER 50 AND 100 CYCLES OF FREEZING AND THAWING

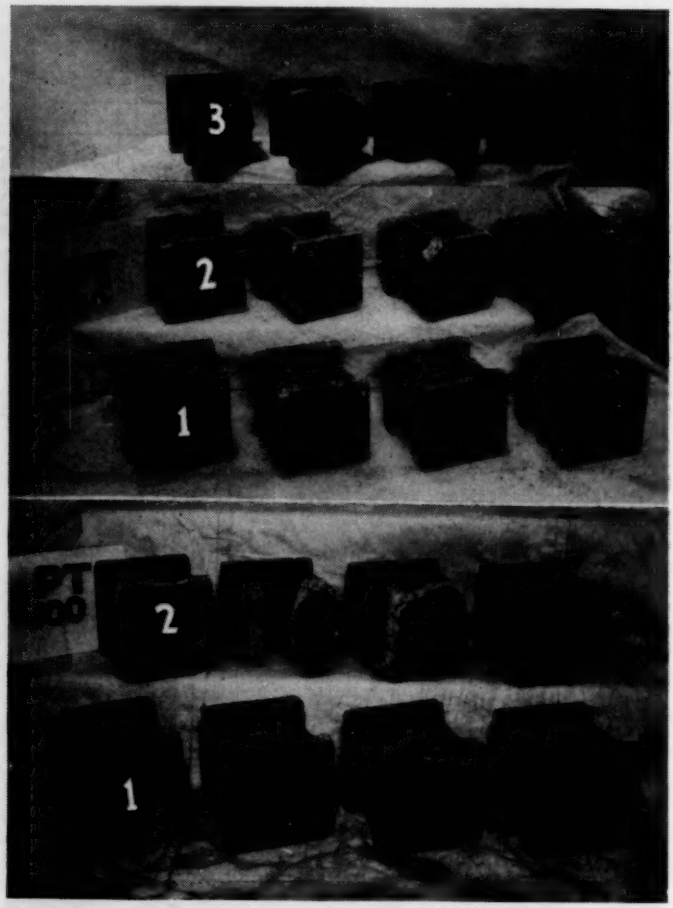


FIG. 19—TAMPED POTTSCO UNITS AFTER 50 AND 100 CYCLES OF FREEZING AND THAWING

weight are shown in Fig. 17. The vibrated units are considerably lighter than the tamped units, absorb less water, and are equal to or slightly greater in strength than the tamped units, except for the 33-unit-per-sack mixes at 270 days.

Durability ratings are "excellent" for mixes V1 and T1, "fair" for mix T2, and "failure" for V2 (Table 12 and Fig. 18 and 19).

The average thermal coefficient of expansion per degree Fahrenheit for the vibrated units was .0000050 and for the tamped units .0000048.

The rate of absorption values are essentially the same for the 3 vibrated mixes and the leanest tamped mix. For the 2 richer tamped mixes, however, these values are considerably lower and it will be

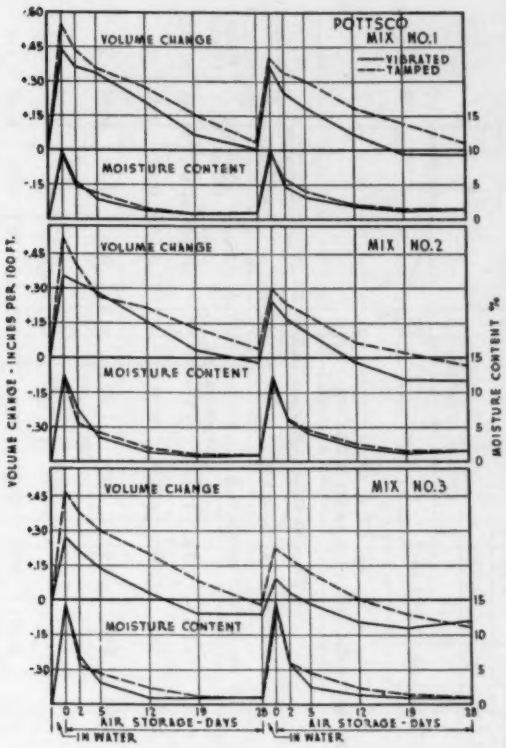


FIG. 20—VOLUME CHANGE DUE TO CHANGES IN MOISTURE CONTENT OF UNIT

noted that the ratio of strength after 100 cycles of freezing and thawing to the strength of the control specimens is above unity in both instances. Suction rate values increase as the richness of mix decreases for the longer periods of immersion but are not quite consistent at early periods.

Sand and Gravel Units

All of the sand and gravel mixes were well above minimum requirements of Federal and A. S. T. M. Specifications for Load Bearing Masonry Units, the lowest 28-day compressive strength and the highest absorption being 1140 p.s.i. and 8.4 lb. per cu. ft. of concrete, respectively, for mix T3. For this material vibration produces denser concrete which is considerably stronger, absorbs less water, and is more resistant to freezing and thawing than tamped concrete. These results are well shown in Tables 14 and 15 and Fig. 21 to 23. The durability rating for all mixes is "excellent."

TABLE 13—PHYSICAL DATA ON SAND & GRAVEL AGGREGATE

Grading	Weight, Lb. per Cu. Ft.		Moisture Content as Sampled	Sieve Analysis % Coarser Than							Fineness Modulus
	Damp Loose	Dry Rodded		⅜	4	8	16	30	50	100	
Fine	106.0	115.5	2.0		3	19	37	54	83	98	2.94
Coarse	104.6	107.0	1.5		52	89	95	97	99	99	5.31
Combined Fine 60% Coarse 40%		120.5	0		23	47	60	71	89	99	3.89

TABLE 14—BATCH DETAILS AND PHYSICAL DATA ON SAND & GRAVEL UNITS

Mix No.	Batch Details					Physical Measurements of Units											
	Proportions per Sack of Cement					Weight of		Compressive Strength*					Absorption		Capillarity**		
								Lb. per Sq. In. Gross Area				Ratio,			Absorption, Lb. per Unit		Water Height in.
	By	Aggregate		Mixing Water, Gal.	Yield, No. of Units	Unit at 28 Da., Lb.	Concrete, Lb./Ft.³	7 Da.	28 Da.	170 Da.	100 F&T Cycles	100 Cy. / 28 Da.	Lb./Ft.³ in 24 Hr.	Ratio, ½ Hr. / 24 Hr.	½ Hr.	24 Hr.	24 Hr.
		Fine	Coarse														
V1	Wt.	407	252	3.7	17.2	45.9	142	2940	3800	3500	3580	0.94	5.2	0.54	0.10	0.25	2
	Vol.	3.45	2.3														
V2	Wt.	606	375	5.0	25.6	44.9	137	1880	2360	2760	2820	1.20	6.5	0.88	0.25	0.40	2
	Vol.	5.15	3.4														
V3	Wt.	800	495	6.9	34.1	44.5	136	1290	1600	1900	2100	1.31	6.9	0.91	0.35	0.55	2½
	Vol.	6.75	4.55														
T1	Wt.	383	235	3.7	17.2	43.1	138	2040	2540	2870	2820	1.11	7.4	0.79	0.25	0.40	1½
	Vol.	3.25	2.15					(0.69)	(0.67)	(0.82)	(0.79)						
T2	Wt.	570	350	5.5	25.3	43.1	136	1390	1800	1950	1880	1.04	7.6	0.82	0.25	0.50	2
	Vol.	4.8	3.2					(0.74)	(0.77)	(0.71)	(0.67)						
T3	Wt.	755	465	5.9	33.2	41.7	132	970	1140	1100	1440	1.26	8.4	0.91	0.50	0.75	3
	Vol.	6.4	4.2					(0.75)	(0.71)	(0.58)	(0.69)						

*Figures in parentheses show strength ratios of tamped to vibrated units at a given age.
**Units set face down in ¼ inch of water.

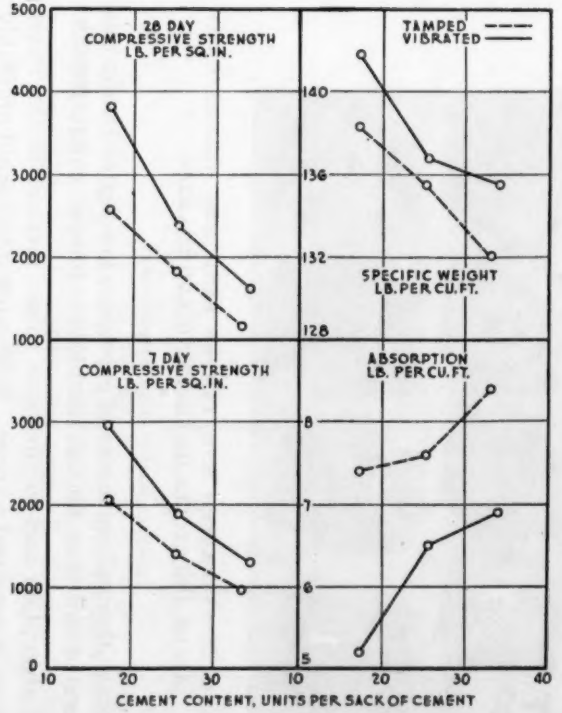


FIG. 21—EFFECT OF METHOD OF MOLDING AND CEMENT CONTENT ON COMPRESSIVE STRENGTH, ABSORPTION, AND SPECIFIC WEIGHT OF SAND AND GRAVEL UNITS

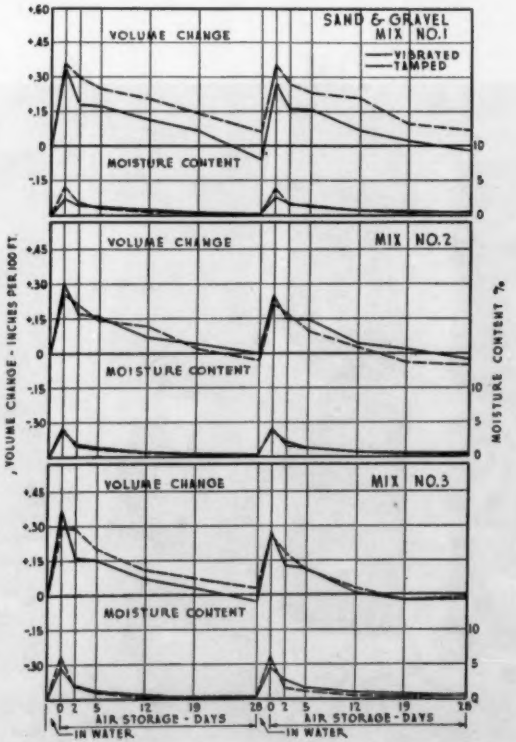


FIG. 24—VOLUME CHANGE DUE TO CHANGES IN MOISTURE CONTENT OF UNIT

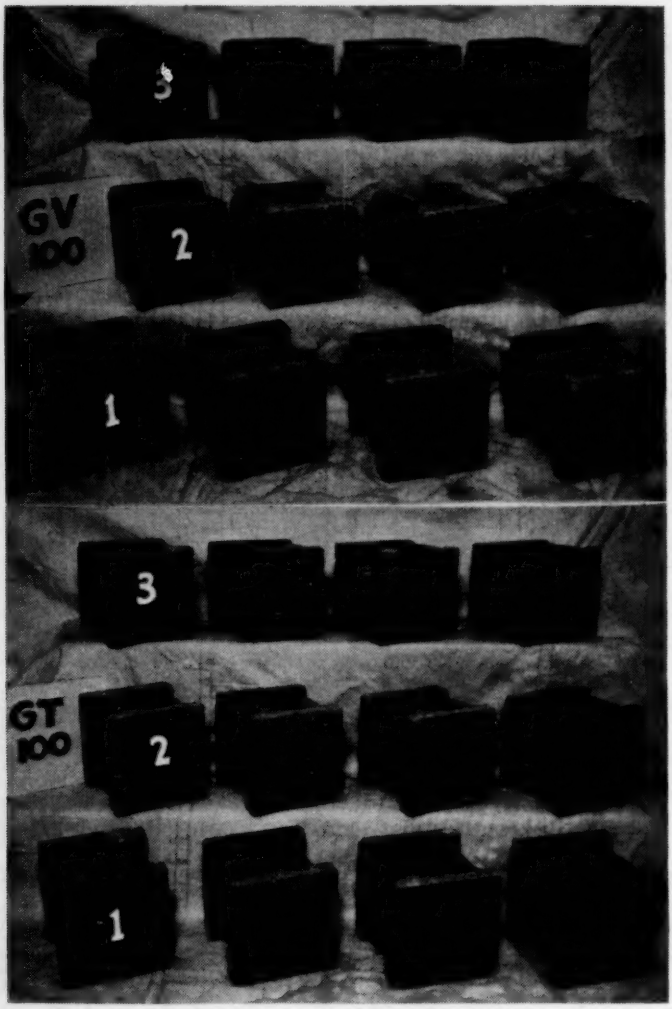


FIG. 22 AND 23—VIBRATED AND TAMPED SAND AND GRAVEL UNITS AFTER 100 CYCLES OF FREEZING AND THAWING

The average thermal coefficient of expansion per degree Fahrenheit for the vibrated units was .0000055 and for the tamped units .0000049.

Rate of absorption and suction rate values increase as the mixes become leaner in a normal manner. Still leaner mixes could profitably have been included to determine limits of strength before disintegration under freezing and thawing might be expected.

TABLE 15—DURABILITY DATA FOR SAND & GRAVEL UNITS

Mix No.	Percentage Change in Weight at Cycle Number		Summary of Observations on Freezing and Thawing Specimens At Cycle Number						Rating
	50	100	15	30	50	65	80	100	
V1	+0.5	+0.4				x	x	x	Excellent
V2	+1.1	+1.1			x	xx	xx	xx	Excellent
V3	+1.1	+1.1			x	xx	xx	xx	Excellent
T1	+0.6	+0.8			x	x	x	x	Excellent
T2	+1.3	+1.0			x	xx	xx	xx	Excellent
T3	+1.2	+1.1	x	x	xx	xxx	xxx	xxx	Excellent

x Very light spalling
xx Light spalling
xxx Light to medium spalling

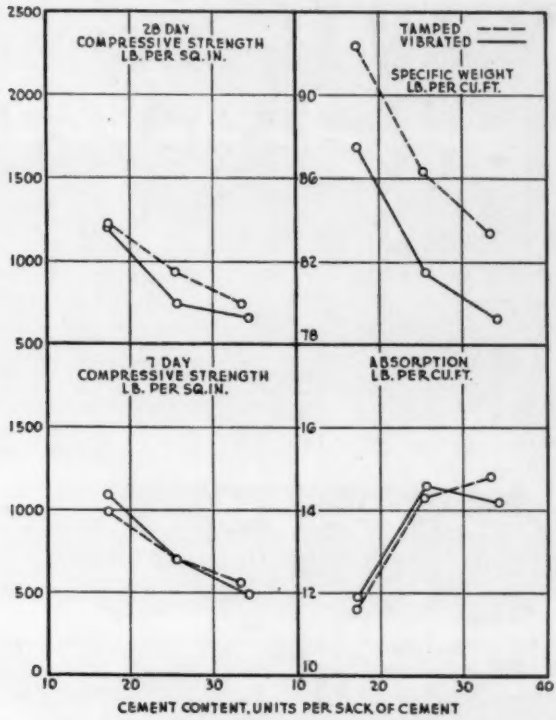


FIG. 25—EFFECT OF METHOD OF MOLDING AND CEMENT CONTENT ON COMPRESSIVE STRENGTH, ABSORPTION, AND SPECIFIC WEIGHT OF SUPEROCK UNITS

TABLE 16—PHYSICAL DATA ON SUPEROCK AGGREGATE

Grading	Weight, Lb. per Cu. Ft.		Moisture Content as Sampled	Sieve Analysis % Coarser Than							Fineness Modulus
	Damp Loose	Dry Rodded		⅜	4	8	16	30	50	100	
Fine	54.4	55.5	8.1		3	14	36	69	89	96	3.07
Coarse	45.4	48.5	11.7	25	69	82	88	92	94	96	5.45
Combined Fine 55% Coarse 45%		57.0		10	30	41	57	78	91	96	4.01

TABLE 17—BATCH DETAILS AND PHYSICAL DATA ON SUPEROCK UNITS

Mix No.	Batch Details					Physical Measurements of Units										
	Proportions per Sack of Cement					Weight of		Compressive Strength*					Absorption		Capillarity**		
	By	Aggregate		Mixing Water, Gal.	Yield, No. of Units	Unit at 28 Da., Lb.	Concrete, Lb./Ft.³	Lb. per Sq. In. Gross Area				Ratio, 100 Cy. / 28 Da.	Lb./Ft.³ in 24 Hr.	Ratio, ½ Hr. / 24 Hr.	Absorption, Lb. per Unit		Water Height in. 24 Hr.
		Fine	Coarse					7 Da.	28 Da.	170 Da.	100 F&T Cycles				½ Hr.	24 Hr.	
V1	Wt.	210	161	5.1	17.2	28.4	88	1080	1220	1350	1290	1.06	11.9	0.87	0.25	0.55	1½
	Vol.	3.5	2.95														
V2	Wt.	312	240	6.9	25.6	26.3	82	700	740	840	440	0.59	14.6	0.90	0.30	0.70	2
	Vol.	5.2	4.40														
V3	Wt.	412	316	9.9	34.1	25.9	79	490	660	580[a]	410[a]	0.62[a]	14.2	0.92	0.40	1.05	3
	Vol.	6.9	5.85														
T1	Wt.	227	172	5.6	17.2	29.9	92	980	1200	1230	1800	1.49	11.6	0.69	0.25	0.70	2
	Vol.	3.8	3.2					(0.91)	(0.98)	(0.91)	(1.39)						
T2	Wt.	338	255	8.3	25.3	28.8	86	710	930	860	1340	1.44	14.3	0.79	0.40	1.00	2½
	Vol.	5.65	4.75					(1.01)	(1.25)	(1.03)	(3.09)						
T3	Wt.	448	339	11.0	33.2	27.4	83	560	740	660	700	0.95	14.8	0.85	0.50	1.45	3½
	Vol.	7.50	6.30					(1.14)	(1.12)	(1.12)[a]	(1.71)[a]						

*Figures in parentheses show strength ratios of tamped to vibrated units at a given age.
[a]Specimens subjected to only 50 cycles of freezing and thawing and control specimens tested at 270 days.

**Units set face down in ¼ inch of water.

Superock Units

All of the Superock mixes except the leanest vibrated mix passed the Federal and A. S. T. M. Specifications for Load Bearing Masonry Units. Fig. 25 shows that vibration produced somewhat lighter units than did tamping; from the standpoint of strength and absorption, however, there is practically no difference between the two. The durability ratings are "excellent" for mixes V1, T1, and T2; "fair" for T3; and "failure" for V2, (Table 18, Fig. 26 and 27). Three of the 4 units of the leanest tamped mix behaved in an excellent manner in the freezing and thawing program. The fourth unit, however,

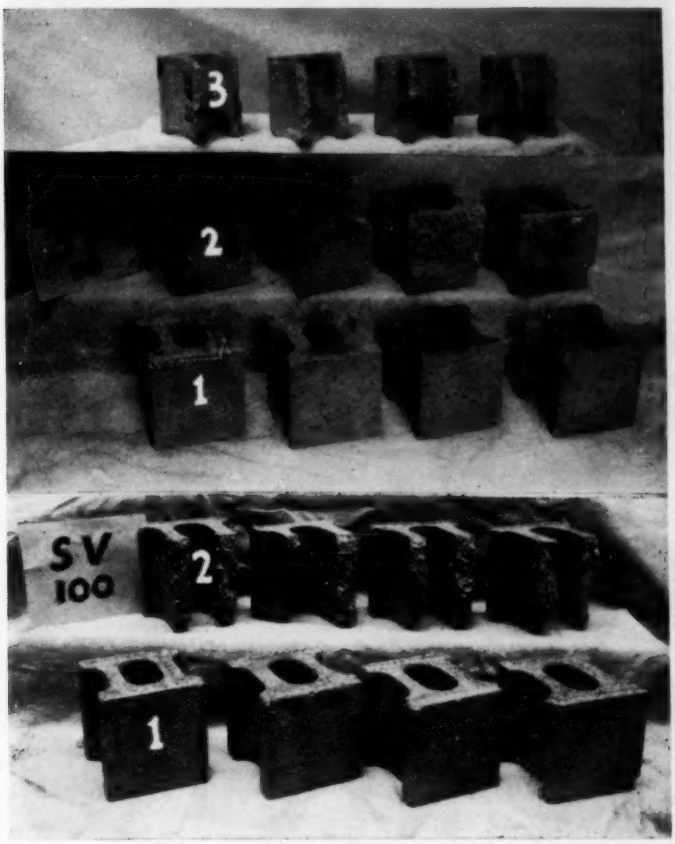


FIG. 26—VIBRATED SUPEROCK UNITS AFTER 50 AND 100 CYCLES OF FREEZING AND THAWING

disintegrated to such an extent that the average weight loss for the group exceeded the permissible value for a rating of "good."

The average thermal coefficient of expansion per degree Fahrenheit for the vibrated units was .0000039 and for the tamped units .0000047.

Rate of absorption values are essentially the same for the 3 vibrated and the leanest tamped mix (Table 17). These values for the 2 richer

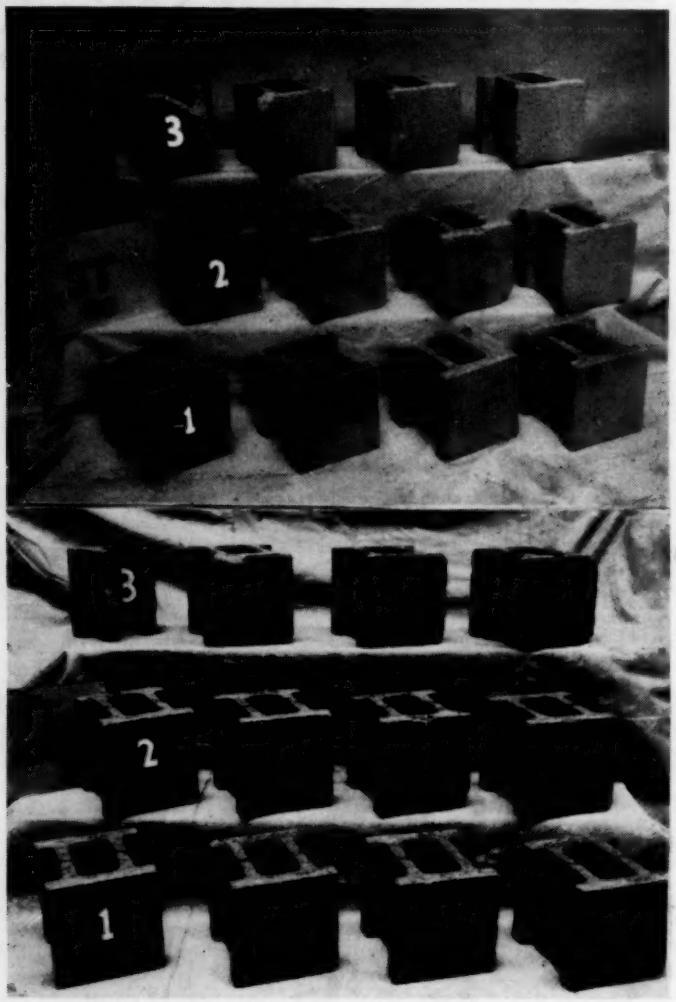


FIG. 27—TAMPED SUPEROCK UNITS AFTER 50 AND 100 CYCLES OF FREEZING AND THAWING

tamped mixes are considerably lower and again it will be noted that for both the ratios of strength after freezing and thawing to the control strength are well above unity. Suction rates for both methods of placement increase as the richness of mix decreases.

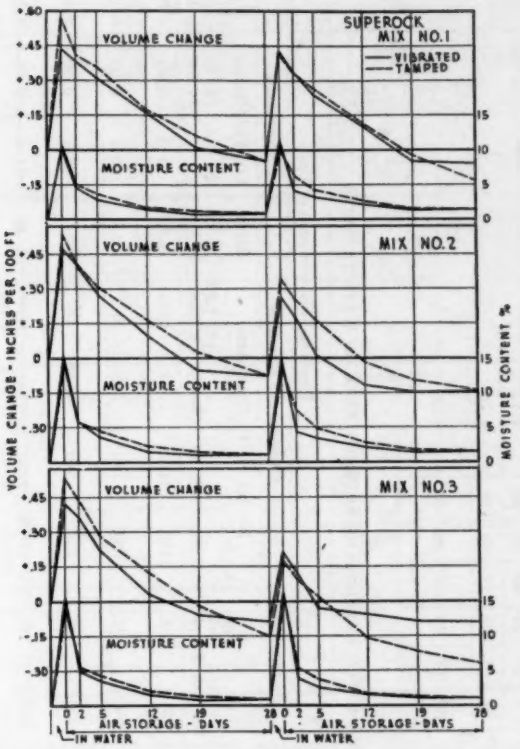


FIG 28—VOLUME CHANGE DUE TO CHANGES IN MOISTURE CONTENT OF UNIT

TABLE 18—DURABILITY DATA FOR SUPEROCK UNITS

Mix No.	Percentage Change in Weight at Cycle Number		Summary of Observations on Freezing and Thawing Specimens At Cycle Number						Rating
	50	100	15	30	50	65	80	100	
V1	+ 1.5	+ 1.3				x	x	xx	Excellent
V2	−11.8	−37.9		*	12%	***	***	38%	Failure
V3	−41.9	50 only	xx	**	41.9%	End of test 50 cycles			Failure
T1	+ 1.1	+ 1.1			x	xx	xx	xx	Excellent
T2	+ 0.5	+ 0.7		x	xx	xxx	xxx	xxx	Excellent
T3	− 1.6	−12.2		x	xxx	*	**	***	Fair

x Very light spalling
xx Light spalling
xxx Light to medium spalling
* Medium spalling
** Heavy spalling
*** Disintegration

TABLE 19—PHYSICAL DATA ON WAYLITE AGGREGATE

Grading	Weight, Lb. per Cu. Ft.		Moisture Content as Sampled	Sieve Analysis % Coarser Than							Fineness Modulus
	Damp Loose	Dry Rodded		⅜	4	8	16	30	50	100	
Fine	42.2	55.5	0	0	3	21	43	69	85	94	3.15
Coarse	42.8	47.7	0	17	81	91	95	97	98	99	5.78
Combined Fine 55% Coarse 45%		58.5		7	36	51	65	81	90	96	4.26

TABLE 20—BATCH DETAILS AND PHYSICAL DATA ON WAYLITE UNITS

Mix No.	Batch Details					Physical Measurements of Units											
	Proportions per Sack of Cement					Weight of		Compressive Strength*					Absorption		Capillarity**		
	By	Aggregate		Mixing Water, Gal.	Yield, No. of Units	Unit at 28 Da., Lb.	Concrete, Lb./Ft.³	Lb. per Sq. In. Gross Area				Ratio, 100 Cy. / 28 Da.	Lb./Ft.³ in 24 Hr.	Ratio, ½ Hr. / 24 Hr.	Absorption, Lb. per Unit		Water Height in. 24 Hr.
		Fine	Coarse					7 Da.	28 Da.	170 Da.	100 F&T Cycles				½ Hr.	24 Hr.	
V1	Wt.	198	146	7.0	17.2	27.9	88	1150	1180	1220	1060	0.90	11.0	0.85	0.25	0.50	1½
	Vol.	3.55	3.1														
V2	Wt.	295	216	10.4	25.5	26.4	84	650	780	790	0	0	12.4	0.93	0.45	0.65	2
	Vol.	5.3	4.6														
V3	Wt.	390	286	13.8	33.0	25.7	82	440	580	525[a]	0[a]	0	12.8	0.90	0.35	0.85	3
	Vol.	7.0	6.1														
T1	Wt.	208	153	7.1	16.9	29.0	90	970	1220	1160	1680	1.38	11.5	0.81	0.30	0.60	2
	Vol.	3.75	3.25					(0.84)	(1.03)	(0.96)	(1.58)						
T2	Wt.	310	228	10.5	24.4	27.6	88	800	990	800	1120	1.13	11.9	0.84	0.45	0.95	3
	Vol.	5.6	4.8					(1.23)	(1.28)	(1.01)	(tamped)						
T3	Wt.	410	300	14.7	32.5	27.2	87	680	920	640	980	1.08	12.7	0.85	0.40	1.05	3
	Vol.	7.4	6.35					(1.55)	(1.57)	(1.21)[a]	(tamped)						

*Figures in parentheses show strength ratios of tamped to vibrated units at a given age.
[a]Specimens subjected to only 50 cycles of freezing and thawing and control specimens tested at 270 days.
**Units set face down in ¼ inch of water.

Waylite Units

All of the Waylite mixes except V3 passed the minimum requirements of the Federal and A. S. T. M. Specification for Load Bearing Masonry Units. Vibrated mixes, Fig. 29, produced lighter units than did the tamped mixes, but absorptions for the 2 methods of placement

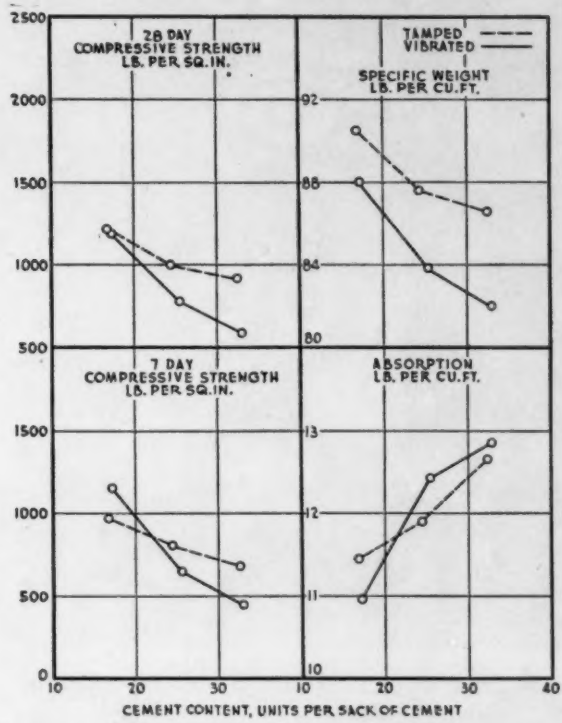

28 DAY COMPRESSIVE STRENGTH LB. PER SQ.IN.
TAMPED
VIBRATED
SPECIFIC WEIGHT LB. PER CU.FT.
7 DAY COMPRESSIVE STRENGTH LB. PER SQ.IN.
ABSORPTION LB. PER CU.FT.
CEMENT CONTENT, UNITS PER SACK OF CEMENT

FIG. 29—EFFECT OF METHOD OF MOLDING AND CEMENT CONTENT ON COMPRESSIVE STRENGTH, ABSORPTION, AND SPECIFIC WEIGHT OF WAYLITE UNITS

TABLE 21—DURABILITY DATA FOR WAYLITE UNITS

Mix No.	Percentage Change in Weight at Cycle Number		Summary of Observations on Freezing and Thawing Specimens At Cycle Number						Rating
	50	100	15	30	50	65	80	100	
V1	+ 1.0	+ 1.3			x	xxx	xxx	*	Good
V2	− 7.6	−72.5	x	xx	8%	***	***	45%	Failure
V3	−50.6	50 only	xxx	**	***	End of test 50 cycles			Failure
T1	+ 1.3	+ 1.4			x	xx	xx	xx	Excellent
T2	+ 1.2	+ 0.8	x	x	*	**	**	**	Excellent
T3	+ 1.6	+ 1.3	x	x	xxx	*	*	**	Excellent

x Very light spalling
xx Light spalling
xxx Light to medium spalling
* Medium spalling
** Heavy spalling
*** Disintegration

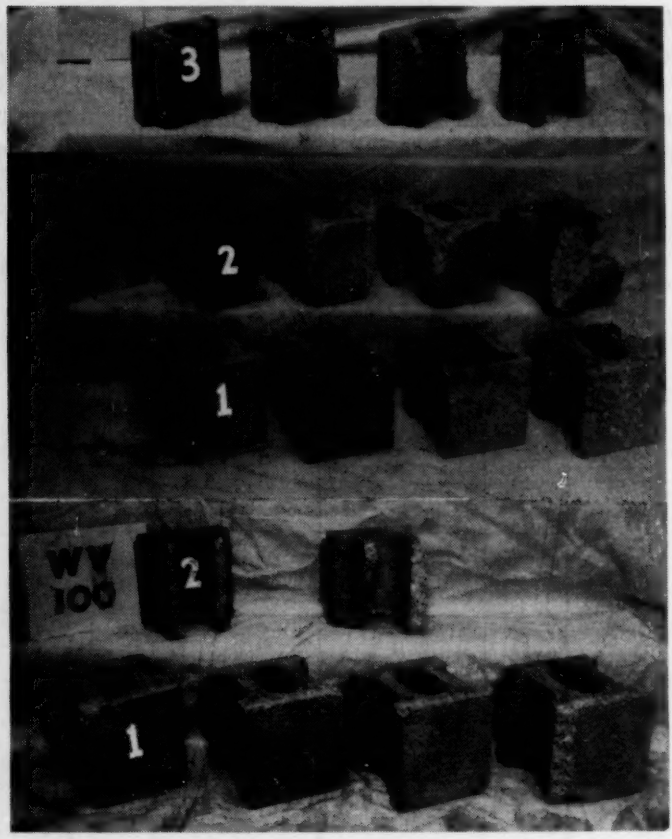


FIG. 30—VIBRATED WAYLITE UNITS AFTER 50 AND 100 CYCLES OF FREEZING AND THAWING

were essentially the same. For the richer mixes the strengths were approximately the same, but for the leanest mixes the tamped units were superior (Table 20). Durability ratings are "excellent" for the 3 tamped mixes, "good" for the richest vibrated mix, and "failure" for mix V2 (Table 21, and Fig. 30 and 31).

The average thermal coefficient of expansion per degree Fahrenheit for the vibrated units was .0000044 and for the tamped units .0000043.

Suction rates (Table 20) for short periods of immersion are inconsistent, but for longer periods increase as the richness of mix decreases. The rates of absorption are slightly lower for the 4 mixes which received a durability rating of good or better, but the correlation is not as perfect as for certain other aggregates.

FIG. 31—TAMPED WAYLITE UNITS AFTER 50 AND 100 CYCLES OF FREEZING AND THAWING

General Notes

The specific weight of concrete in all instances decreased as the mixes became leaner. Tamping, except for sand and gravel, produced somewhat denser concrete than did vibration, the average specific weight considering all mixes and aggregates being about 4 per cent higher. On the other hand, the tamped units absorbed more water except for Superock, where absorptions for both methods of placement were

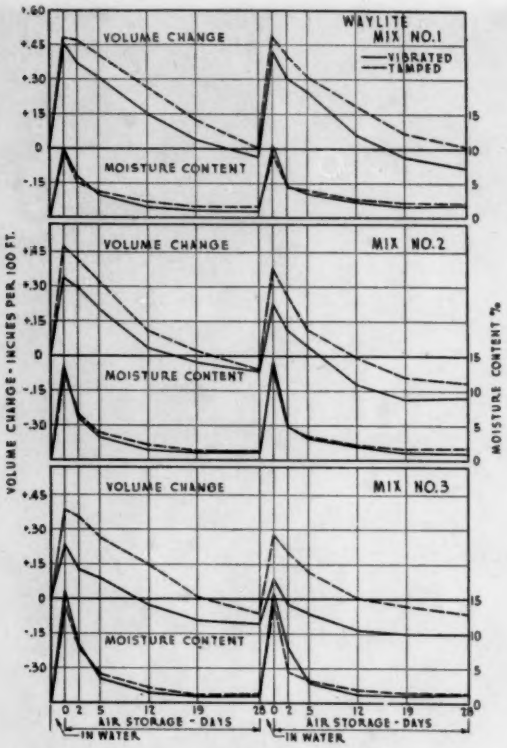


FIG. 32—VOLUME CHANGE DUE TO CHANGES IN MOISTURE CONTENT OF UNIT

equal, and for limestone, where the vibrated mix was not satisfactory. In all cases the absorption increased as the mixes became leaner, but in no case did the total absorption exceed 14.8 lb. per cu. ft. of concrete —under the 15 and 16 lb. per cu. ft. permitted in the governing specifications.

Volume changes closely followed changes in moisture and for equal moisture contents were proportional to the cement content (Fig. 8, 12, 16, 20, 24, 28 and 32). Tamped units exhibited slightly greater volume changes than did vibrated units except for Haydite where no essential difference was noted. This was generally most marked in the leanest mixes. A second cycle of wetting caused the richest mixes for each aggregate to expand to almost the same extent as initial wetting. The lean mixes, on the other hand, expanded to a smaller degree, finally showing shrinkages from original values which made the total volume change of about the same magnitude as obtained for the first wetting cycle.

Vibratory methods of placement produced stronger units at 7 days of age than did tamping for 14 of the 21 mixes, the average strength of the vibrated units being 5½ per cent greater than for tamped. At 28 days vibrated units were still stronger in eleven instances, but the average strength was 4 per cent greater in favor of tamping; at 6 months vibrated units were 9½ per cent stronger than tamped units, the latter showing superiority in only 8 cases. If limestone, for which the vibrated mixes were unsatisfactory, is excluded, the percentages become 9, 1, and 16½, all in favor of vibration. It is of interest to note that for all mixes of cinder, Haydite, and sand and gravel units the 7-day vibrated strengths were essentially equal to the 28-day tamped strengths.

Seven- to 28-day strength ratios were 0.78 for tamped and 0.85 for vibrated units, the latter gaining a much larger proportion of strength at an early age than the former. After 5 months additional laboratory storage in ordinary air, the average strength of the tamped units had not changed materially. The 7- to 170-day strength ratio dropped to only 0.77. In 4 instances there were strength decreases over 10 per cent compared with 28-day values, occurring for the leanest cinder and Superock mixes and both leaner Waylite mixes. This condition did not develop with vibrated units, although the leanest Waylite and Superock units lost slightly in strength, and as a consequence the 7- to 170-day strength ratio for all vibrated mixes dropped to 0.78. While little strength gain after 28 days is to be expected for units cured and stored as in this program, neither are large strength losses; but no apparent reason can be supplied for the unusual behavior in the 4 cases especially noted above.

For the few mixes where serious spalling occurred in the tamped units during the durability tests it began in the face shell immediately adjacent to the junction of the face shell and one of the webs. The action was then progressive through the web and one half of the exposed face before serious attack occurred in the other half of the unit. Note Fig. 7, 15 and 19 for examples of such behavior. It was not possible to determine whether this defect always occurred in the same position of the unit as manufactured or not, but if this was the case, the failure could possibly have been due to minor misadjustment of feed or tamping action.

A peculiar type of failure was noted for the two leaner vibrated mixes of all slag aggregates. Fairly early in the program, the exact time depending upon the strength of the unit, face shell cracks were noted which progressed through the unit, and finally resulted in the spalling off of the entire face to approximately one-half the thickness

of the face shell. The condition is well illustrated in Fig. 33, a unit from mix V2 of Superock after 50 cycles of freezing and thawing. No satisfactory explanation for this behavior has appeared, the condition being limited to slag aggregates and vibrated units and not being apparent in any tamped block nor any aggregate other than slag.

FIG. 33—FACE SHELL SEPARATION TYPICAL OF V2 AND V3 MIXES MADE WITH SLAG AGGREGATES

A general summary of durability results appears in Table 22. Aggregates are listed in order of unit strengths after 100 cycles of freezing and thawing. It will be noted that no single criterion is entirely satisfactory for rating these materials from the durability standpoint, nor is the combination of strength and absorption used in current specifications, even when modified as to specific values completely satisfactory. If a minimum strength of 1000 p.s.i. at 28 days is used as a criterion, 4 of the 21 mixes rated "excellent," all "fair," and all but one failure mix (cinder V2) would fail to pass the specification. A promising combination of properties for predicting durability is a rate of absorption specification coupled with a 28-day compressive strength requirement. For the material included in this program, a 28-day strength of 900 p.s.i. based on gross area, and a maximum rate of absorption of 0.87 based on ½ hour and 24 hour immersion rules out all "fair" and "failure" mixes, but also four "excellent" mixes. The latter are the 25- and 33-unit-per-sack vibrated sand and gravel, the 33-unit-per-sack tamped sand and gravel, and the 17-unit-per-sack vibrated Pottsco mixes. In all four cases it is the rate of absorption specification which causes rejection. The suggested combinations of strength and rate of absorption, however, result in a

TABLE 22—GENERAL SUMMARY OF DURABILITY RATINGS

Mixes arranged in order of compressive strength after 100 cycles

Mix No.	Compressive Strength				Absorption		Weight Change After 100 Cy. Per Cent	Durability Rating
	Lb. per Sq. In.		Ratio		Lb./Ft.³ 24 Hr.	Ratio ½ Hr. / 24 Hr.		
	After 100 Cycles	At 28 Days	100 Cy. / 28 Da.	100 Cy. / 170 Da.				
GV1	3580	3800	0.94	1.02	5.2	0.54	+0.4	Excellent
GV2	2820	2360	1.20	1.02	6.5	0.88	+1.1	Excellent
GT1	2820	2540	1.11	0.98	7.4	0.79	+0.8	Excellent
LT1	2190	1940	1.13	0.93	11.9	0.64	−0.7	Good
GV3	2100	1600	1.31	1.10	6.9	0.91	+1.1	Excellent
HV1	2100	1810	1.16	1.00	7.9	0.59	+1.0	Excellent
HT1	1980	1450	1.36	1.29	12.1	0.64	+1.6	Excellent
GT2	1880	1800	1.04	0.96	7.6	0.82	+1.0	Excellent
CT1	1880	1610	1.16	1.22	11.3	0.72	+0.1	Excellent
ST1	1800	1200	1.49	1.46	11.6	0.69	+1.1	Excellent
WT1	1680	1220	1.38	1.44	11.5	0.81	+1.4	Excellent
PT1	1620	1210	1.34	1.23	13.2	0.67	+1.1	Excellent
CV1	1590	1680	0.94	0.84	10.0	0.86	+1.6	Good
HV2	1480	1280	1.16	0.97	13.1	0.87	+2.1	Excellent
PV1	1440	1210	1.20	0.92	11.9	0.88	+1.3	Excellent
GT3	1440	1140	1.26	1.31	8.4	0.91	+1.1	Excellent
HT2	1360	1170	1.16	1.24	13.6	0.72	+1.2	Excellent
HT3	1340	970	1.39	1.34	14.2	0.76	+1.9	Excellent
ST2	1340	930	1.44	1.56	14.3	0.79	+0.7	Excellent
SV1	1290	1220	1.06	0.96	11.9	0.87	+1.3	Excellent
HV3	1250	1150	1.09	0.89	13.7	0.84	+1.1	Excellent
PT2	1190	890	1.34	1.08	13.8	0.74	−7.8	Fair
WT2	1120	990	1.13	1.41	11.9	0.84	+0.8	Excellent
WV1	1060	1180	0.90	0.87	11.0	0.85	+1.3	Good
WT3	980	920	1.08	1.55	12.7	0.85	+1.3	Excellent
ST3	700	740	0.95	1.07	14.8	0.85	−12.2	Fair
LV1	700	1180	0.59	0.44	13.4	0.91	−20.1	Failure
CT2	640	970	0.66	0.55	12.2	0.90	−24.5	Failure
PV2	480	940	0.52	0.40	13.2	0.89	−47.3	Failure
SV2	440	740	0.59	0.52	14.6	0.90	−38.0	Failure
LT2	220	930	0.23	0.17	12.6	0.91	−81.0	Failure
CV2	0	1040	0	0	12.1	0.88	−36.4*	Failure
LT3	0	920	0	0	12.1	0.88	−60.6*	Failure
LV2	0	830	0	0	14.0	0.92	−66.6*	Failure
WV2	0	780	0	0	12.4	0.93	−72.5	Failure
CV3	0	730	0	0	12.1	0.88	−38.6*	Failure
LV3	0	720	0	0	13.5	0.92	−82.9*	Failure
CT3	0	700	0	0	12.9	0.92	−47.1*	Failure
PT3	610*	660	0.92*	0.69*	14.3	0.84	−19.1*	Failure
SV3	410*	660	0.62*	0.70*	14.2	0.92	−41.9*	Failure
PV3	270*	600	0.45*	0.42*	14.2	0.89	−37.7*	Failure
WV3	0*	580	0*	0*	12.8	0.90	−50.6*	Failure

*After 50 cycles of freezing and thawing.

division which is too finely drawn to be usable in a general specification without considerably more supporting data, since small changes in these values would rule out many "excellent" units or permit many unsatisfactory units to pass. In this connection it is entirely possible that minor changes in the grading of aggregate used, particularly the proportion passing the 50 mesh sieve, would have a marked influence on the rate of absorption. It seems entirely probable that further work in determining rate of absorption for other time inter-

vals, possibly with a boiling period included, would provide a more positive means for predicting performance under severe exposure conditions.

CONCLUSIONS

While the scope of this program was confined to a single grading for each aggregate and included only one type of cinders, limestone, and sand and gravel, the following statements seem warranted:

1. For equivalent yields with the same aggregate, vibration produces units which have higher early strengths and absorb slightly less water than does tamping. The units produced by vibration are also lighter except for the sand and gravel mixes.

2. Compressive strength and specific weight decreased and absorption increased as yield in terms of units per sack of cement increased in approximately linear relationship for each individual aggregate in the program.

3. The thermal expansion coefficient per degree Fahrenheit varied with the type of aggregate from a low of .0000021 for cinders to a high of .0000055 for sand and gravel. In general, there is no marked difference in this coefficient for different yields with the same aggregate or for method of molding.

4. Volume changes closely follow changes in moisture and are essentially proportional to the cement content.

5. Tamped units in most instances exhibit slightly greater volume changes than do vibrated units for equivalent changes in moisture.

6. A larger proportion of the tamped units than of the vibrated proved durable under 100 cycles of freezing and thawing.

(a) With yields of 17 block per sack, 13 of 14 mixes, 6 vibrated and 7 tamped, produced satisfactory durability ratings.

(b) With yields of 25 block per sack, 7 of 14 mixes, 2 vibrated and 5 tamped, received fair or better durability ratings.

(c) With yields of 33 block per sack, 6 of the 10 mixes with strengths over 700 p.s.i., 2 vibrated and 4 tamped, produced durability ratings above failure.

7. The average strength after 100 cycles of freezing and thawing for the 26 mixes which rated above failure was 18 per cent higher than the 28 day strength and 14 per cent higher than the strength of the control specimens tested at a comparable age.

8. On the basis of these tests good or excellent resistance to 100 cycles of freezing and thawing is assured if the units show no weight loss at the end of 50 cycles.

9. Present specifications of 700 p.s.i. compressive strength at 28 days and a maximum absorption of 15 or 16 lb. per cu. ft. of concrete

are inadequate to insure masonry units which will be sound and durable under severe exposure conditions.

10. There is no apparent correlation between absorption per se and durability. Rate of absorption, however, expressed as the ratio of absorption after ½ hour immersion to absorption after 24 hours immersion in water, shows promise as a criterion for durability rating.

11. A satisfactory specification based on these tests which insures excellent resistance to freezing and thawing for light weight aggregates is a compressive strength at 28 days of 900 p.s.i. based on gross area and a rate of absorption not in excess of 0.87. The latter measure, however, requires further confirmation before it can be recommended for use in specifications.

12. A minimum compressive strength of 1000 p.s.i. of gross area is recommended for load bearing units as the simplest and most equitable specification for insuring reasonably satisfactory behavior under severe exposure conditions.

13. Unsound aggregates, as evidenced by the particular limestone used in these tests, cause unsatisfactory behavior of units under severe exposure despite conformance to reasonable initial strength specifications.

14. Further studies should include the effect of differences in the proportion of aggregate passing the 50 mesh sieve on absorption rate, strength and durability; and possible correlation of various methods of determining absorption rate with strength as a measure of durability.

For expanded slag aggregates, the effect of variations in the methods of vibration molding and the influence of variations in grading on durability should also be investigated.

Discussion, to close in April, 1940 JOURNAL, *should reach A. C. I. Secretary in triplicate by Feb. 1, 1940.*

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(Supplement)

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AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN *SEPTEMBER 1940*

Discussion of a Paper by Wendt and Woodworth:

Tests on Concrete Masonry Units Using Tamping and Vibration Molding Methods*

BY HARRY C. SHIELDS, BENJAMIN WILK, J. C. PEARSON,
M. O. WITHEY, D. E. PARSONS AND AUTHORS

CONVENTION DISCUSSION

H. C. Shields†—From the valuable data presented, durability ratings of the test specimens were based on: (1) weight loss; (2) ratio of strength after 100 cycles of freezing and thawing to strength at 28 days; (3) ratio of strength after 100 cycles of freezing and thawing to strength at 170 days.

To merit a rating of "excellent," no loss of weight was permissible and units were required to show a minimum strength ratio of either 0.95 at age 28 days or 0.90 at 170 days. The rating was "good," when weight loss did not exceed 2 per cent and strength ratios were over 0.90 or 0.85 respectively. Rating was "fair" when disintegration was confined to a single specimen; the average weight loss less than 15 per cent and strength ratios were within limits prescribed for a "good" rating.

From the data presented, I concur with the authors that *no single* criterion is entirely satisfactory for rating the aggregates from a durability standpoint nor is the combination of strength and absorption used in current specifications, even when modified as to specific values, completely satisfactory.

The factors that appear to have influenced durability ratings of test specimens may be summarized as follows:

*JOURNAL Am. Concrete Inst., Nov. 1939, *Proceedings* Vol. 36, p. 121.
†Central Service Manager, Marquette Cement Mfg. Co.

1—For equivalent yields with the same aggregate, (a) Type of molding equipment. A larger proportion of the tamped units than of the vibrated units proved durable under 100 cycles of freezing and thawing.

2—Loss of weight at end of 50 and 100 cycles of freezing and thawing.

3—Rate of absorption expressed as the ratio of absorption after ½ hour immersion to absorption after 24 hours immersion in water.

4—Compressive strength at age 28 days, based on gross area.

5—Unsound aggregates

6—Cement factor, (a) Yield in terms of units per sack of cement.

Of the six factors tending to influence durability, the last named, the cement factor or yield, appears to have played the major role in durability performance of all the test specimens.

It will be observed that: (1) Absorption increased in approximately linear relationship for each type of aggregate, as yield increased or cement factor decreased. (2) The rate of absorption was influenced by cement factor or yield. The rate being lowest for the rich mixes, and the durability *percentage* factor, the highest. (3) High compressive strengths attended low yield and high cement factors. From Table 22, the mixes arranged in descending order of 28 day compressive strength, i.e. from high to low, from a maximum of 3800 p.s.i. to 920 p.s.i. had 21 durability ratings of "excellent"; and 3 "good" out of a total of 25 mixes. From a maximum of 1180 p.s.i. to a minimum of 580 p.s.i. there were 16 ratings of "failure" and 1 "fair" out of 17 mixes.

In the entire realm of cement usage, the quality of the cement paste is universally recognized as being one of the greatest factors influencing the service performance of any type or form of concrete be it structural, masonry or ornamental.

I do not wish to convey the impression that rich mixtures, or mixtures of high cement factor are the sole means of producing durable concrete, but I do maintain that sufficient cement to insure proper quality of paste is essential to the end of securing durability in any type of concrete.

To modify specifications, based on the data presented in the authors' paper to the extent of requiring a 28-day compressive strength of 900 p.s.i. based on gross area and a rate of absorption not in excess of 0.87, for the purpose of insuring excellent resistance to freezing and thawing, is most certainly a step in the right direction but—it does not go far enough and unfortunately would penalize the product manufacturer to the extent of taking him out of competition with

clay and other products, in many instances. The economics of imposing such requirements by specification to apply to *all* masonry units, is unsound and unnecessary.

There exists a potential market for millions of masonry units to be used for back-up, partitions, floor fillers, etc. (units that will not be exposed to the elements or to freezing and thawing). For such usage provision should be made by specification requirements for the use of masonry units having a lower durability factor; lower compressive strength, which can be made with a lower cement factor than units which are to be used in exposed masonry of the heavy load bearing type.

This could be accomplished best by modifying current specifications so that concrete masonry units could be furnished in two classifications—(1) for load bearing concrete masonry units exposed to the elements—a minimum 28 day compressive strength of 1000 p.s.i. and a rate of absorption not in excess of 0.87; (2) for non-exposed masonry i.e., concrete masonry units used for back up, partitions, fireproofing, floor filler, etc.—a minimum 28 day compressive strength of 700 p.s.i., absorption not to exceed 15 or 16 pounds per cubic foot of concrete.

*Benjamin Wilk**—With a prepared discussion in front of me, Mr. Shields' remarks stir up a few other thoughts, something along this line, of increasing the demand for strength to, say, a thousand pounds per square inch. I can see that, in severe exposures we might require 1000 p.s.i., but Mr. Shields brings up the point of competition. It means added cost if you go to a thousand pounds. It is not very difficult to get 1000 p.s.i. in 28 days, but I dare say that, in the large centers very few blocks are delivered on the job at 28 days. By the use of high early strength cement, we get 1000 lb. strength in much less time. We have been operating under the 700 p.s.i. requirement for many years and I question how many structures have deteriorated as a result of not having the 1000 lb. strength at 28 days. It might be all right to specify 1000 p.s.i. at 28 days but I certainly would not want the specification to add the words "or at the time of delivery." Such a provision would penalize the products manufacturer.

The question arises, what relationship is there between a hundred freezings and thawings and actual exposure. I wish to emphasize that 700 p.s.i. at time of delivery has been very satisfactory. We may want to get away from that but we don't want to say 1000 p.s.i. at the time of delivery. Now for my prepared discussion:

*General Manager, Standard Building Products Co., Detroit.

From the standpoint of the concrete products manufacturer, the first reaction to the paper is a keen appreciation of the work done in arriving at facts that make possible effective thinking on the part of the manufacturer. The question, whether or not to vibrate concrete, in the making of concrete products, has been in the mind of the products manufacturer for several years due to the work that had already been done on vibrating in other types of concrete.

The paper opens the eyes of products manufacturers to the possibilities of vibration, especially in the manufacture of units using sand and gravel as aggregate, in which a considerable increase in strength and lower absorption was obtained with vibration. The seven-day vibrated strengths were practically equal to the 28-day tamped strengths. Volume change was less in the vibrated units than in the tamped units. Even at 33 block per sack the vibrated units had less than 7 per cent absorption and exceeded 1000 p.s.i. at 7 days. A products manufacturer who can consistently produce a marketable unit with 33 block per sack with less than 7 per cent absorption and 1000 p.s.i. strength has accomplished something for the building industry. The products manufacturer must realize that these results were obtained with careful supervision, accurate aggregate and curing control—items which may not be common to most products plants. But the results are something for the products manufacturer to strive to achieve. He can see now what can be accomplished.

Due to the magnitude of the test program it was impossible to use more than one grading in comparing tamping with vibrating. It occurs to me that the grading best for tamping may not be the best for vibrating. In the tests under discussion the percentage of sand and gravel aggregate coarser than ¼ in. was 23 per cent. This combined with the fine aggregate gave a fineness modulus of 3.89 which would seem very good for tamping. In the vibrating process the percentage of material coarser than ¼ inch could probably be a great deal more than 23 per cent and the increase in strength due to the coarser aggregate should be considerable. It is noticeable that the vibrated specimens are heavier than the tamped specimens. With the increased strength obtained in vibrating it should be a simple matter to increase the size of the cores, cut down the wall thickness thereby, and obtain a weight per unit approximately the same as for the tamped unit. The tendency throughout the industry is to develop lighter weight units and if high strengths can be obtained through vibrating, weights could be reduced so as to make the units more acceptable to the masonry contractors.

The authors are to be complimented for presenting the details in such a way as to make possible a careful study of the results. In looking over Table 14, page 146, I am wondering how it happens that only 5.9 gal. of mixing water was used in the batch where 33 tamped units were obtained whereas 6.9 gal. were used in the comparable vibrated batch. There is an increase of only 0.4 gal. between the 25 block per sack batch and the 33 block per batch in the tamped units but an increase of 1.9 gal. per batch between the vibrated 25 block per sack and 34 block per sack units.

Manufacturers using various aggregates will have to study the results in detail to obtain the greatest value from the study as the results are not uniform for all types of aggregates. The data are guide posts for the progressive manufacturer. Further tests by individual manufacturers who may have both vibrating and tamping equipment in their plants ought to be made to supplement the information in this paper. The tests under discussion were made in two plants—one for vibrating, the other for tamping. Comparisons under one roof, with one set of operators, one class of aggregate, with one curing room ought to be even more enlightening than the tests under discussion.

The concrete products industry is to be congratulated upon the cooperative attitude on the part of the producers of concrete products machinery and the producers of various kinds of aggregates. It was apparent at the beginning that conflicting claims would be more or less settled by the project and the concrete products industry is deeply indebted to the machinery manufacturers and the aggregate producers for making possible such an enlightening series of tests.

*J. C. Pearson**—Up to recent years, I have always thought of a unit of this type as going into a structure which would be protected. Since I built a cinder concrete block house with stucco on it in Allentown some years ago, I have been interested in the selection of materials and the design. I think if I could have picked the block, I would have preferred one of 500 p.s.i. rather than 1000 p.s.i. The question of strength was not under consideration but the question of conductivity was, and certainly I am not ready to support a universal recommendation for a thousand pound block. I think a weaker and more porous block would be better, at least for walls that can be protected.

M. O. Withey†—This discussion will be confined to a brief consideration of experiences which we have had at the University of

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†Professor of Mechanics, University of Wisconsin, Madison.

Wisconsin during the last 30 years in attempting to secure an accelerated freezing and thawing test which would correlate with the slower procedure of nature in this climate. Our first weathering tests of importance on concrete, begun in 1910† are still in progress along with five other series of tests, most of which have been reported to this Institute.‡

In the tests of 1910 the concrete was of 1:2:4 and 1:3:6 proportions and made with a coarsely ground low C_3S portland cement and a soft lower magnesian limestone aggregate. Cylinders of that concrete resting on well-drained soil out of doors over a 20-yr. period, judging from our recent thermometer records, must have suffered more than 500 cycles of freezing and thawing, yet they exhibited a progressive increase in strength during that period. However, when 3 cylinders of each class of concrete used in this series were immersed in water and subjected to freezing and thawing at the Portland Cement Association Laboratory in 1932, only 20 to 40 cycles of a medium rate of freezing were required to produce 10 per cent or more loss in weight. Since the remainder of the 1910 specimens still appear as sound as in 1930 when last tested, the information from these tests indicates that one cycle of the accelerated test applied by the P.C.A. laboratory in 1932 was equivalent to at least 20 freezings under the conditions nature imposed on these specimens.

Later experiments convinced us that the flexure test was a more sensitive index of damage by freezing and thawing than the crushing test and that either is far more sensitive than the loss in weight or expansion. Observations of the behavior of Mattimore's walletts embedded in the wet ground of the Alleghany plateau near Kane, Pennsylvania, coupled with some beam tests of limestone concrete, made in our laboratory by L. O. Hanson, pointed to the drastic action produced by freezing and thawing a beam or slab with its lower surface immersed and upper surface free. Such specimens so treated were used in the tests of the Highway Research Board Project Committee on Durability of Concrete as Affected by the Cement. In this program the Pennsylvania State Highway Materials Laboratory thawed their 1:2 mortar prisms in air and thereby caused a more rapid break down of the specimens than was obtained in other participating laboratories

†"Some Long Time Tests of Concrete," M. O. Withey, JOURNAL Amer. Concrete Inst., Feb. 1931; *Proceedings* Vol. 27, p. 547.

‡"Freezing and Thawing, Permeability and Strength Tests on Vibrated Concrete of Low Cement Content," M. O. Withey, JOURNAL Amer. Concrete Inst. May-June 1935, *Proceedings* Vol. 31, p. 528. "Factors Affecting the Resistance to Freezing and Thawing of Vibrated Concrete Made of Crushed Dolomite," M. O. Withey, JOURNAL Amer. Concrete Inst. June 1939, *Proceedings* Vol. 35, p. 553. "Weathering Tests on Concrete," L. O. Hanson, *Proceedings* Highway Res. Board, Dec. 1933, p. 303. "Report of Project Committee on Durability of Concrete as Affected by the Cement," *Proceedings* Highway Res. Board, Nov. 1936, p. 135. "Comparison of The Physical and Mechanical Properties of Hand-rodded and Vibrated Concretes Made with Different Cements," by G. W. Washa, JOURNAL Amer. Concrete Inst. June 1940; *Proceedings* Vol. 36, p. 617.

using comparable freezing rates in their tests. Fortunately at Wisconsin we had made additional specimens which after moist curing for 3 months were placed half immersed in shallow metal trays and subjected to weathering out of doors. In the summer, water was occasionally added to these trays. After 5½ years exposure during which a thermometer in a 6 by 12-in. cylinder recorded 146 cycles of freezing and thawing, the tray of 1:4½ mortar specimens evinced many failures, and many of the 1:2 mortar prisms also exhibited signs of incipient failure. Many of these prisms were bent due to the greater elongation of the lower (immersed) portion of the specimen.

Strength tests on these specimens have not yet been made. Basing judgment on the loss in weight, expansion, and appearance of the 1:2 prisms and the data obtained by the Pennsylvania Highway Laboratory in their accelerated tests it appears that specimens subjected to weathering in this climate with lower portions immersed will fail under about the same number of cycles as in this type of accelerated laboratory test conducted at a medium rate of freezing with partially immersed specimens.

From these data it appears that the freezing and thawing test used by Messrs. Wendt and Woodworth is more drastic than that applied to the 1910 cylinders by the P.C.A. Laboratory. Hence in a wall exposure, where the ground is dry or the ground water is prevented from ascending into the wall by a suitable cutoff plate or membrane, blocks which exhibit *good* and *excellent* ratings in these accelerated tests should withstand natural cycles equivalent to a 50-year period of service. As the tests demonstrate, partial immersion of such units in water during freezing will materially cut down the life. The magnitude of this destructive effect will depend upon the severity of the exposure. In courses near the ground line of building walls under worst conditions the exposure may be sufficiently severe to produce surface disintegration in five to ten years.

Since our thermometer records show that in the vicinity of the freezing point of water the temperature rarely drops or rises at a faster rate than 3° F. per hour, we have preferred to use a comparable rate of freezing in accelerated tests. In the refrigerator at the University of Wisconsin Materials Laboratory the temperature of a 2 by 2 by 9-in. prism is lowered from 35 to 15° F. in 5 to 7 hr.

The thawing in air while slower than thawing in water is more severe and simulates common types of exposure. The combination of freezing and thawing action in such tests appears to produce severe alternations of bending stresses which accelerate the break-down of the specimens.

*Mr. Parsons**—If one attempts to visualize the several operations of the investigators, beginning with the selection and shipping of the materials and ending with the testing of the blocks at the laboratories and the preparation of the report, he must be impressed by the magnitude of the task which confronted the authors. This comprehensive investigation helps to solve several problems of interest to the manufacturers and users of concrete blocks, but it also raises several questions. With some of the aggregates the blocks molded with the vibrating equipment seemed to be superior, whereas with others the tamped units were superior. The reasons are not apparent. There is a possibility that differences in grading accounted for some of the variations in performance and that a further study will show the desirability of varying the grading with the method of molding.

The extent to which the resistance to freezing and thawing may be of importance in a particular construction depends upon the climate, the type of structure and the location of the units in the structure. The cost of replacing defective units is so great that a high resistance to frost action is desirable whenever there is a possibility of severe exposure. The high resistance to freezing and thawing shown by some of the blocks of lean and porous concretes indicates that commercial products may be more resistant than would be predicted by comparison with the performance of structural concrete of similar physical properties. Of course it is disappointing to learn that none of the simple measures for predicting resistance to frost action correlated closely. Although absorption long has been one of the properties limited in specifications, no useful relation between absorption and frost resistance was found. Compressive strength seems to remain the best simple measure yet found for judging frost resistance. As pointed out by the authors, the ratio of the absorption by ½-hour immersion to that at 24 hours may be a useful guide to manufacturers, but is too sensitive for use in specifications. In addition to the factors specifically discussed by the authors, the poor grading of the limestone and cinder aggregates may have contributed to the unusually poor showing of the blocks made with these materials. A large proportion of the limestone aggregate was between the No. 4 and No. 8 sieves and an unusually high percentage of the cinder aggregate was of particles between the No. 8 and No. 16 sieves.

The search for a simple, rapid and reliable method for estimating the durability of concrete products should be continued. Possibly it would be worthwhile to make further comparisons between the saturation coefficient and resistance to frost action. The saturation coeffi-

*National Bureau of Standards, Washington, D. C.

cient, which is a measure of the proportion of the total fillable pores which are saturated by simple soaking in cold water, has been found to be a useful guide in the selection of clay products.[1] Recent investigations of clay products indicate that more reliable estimates of the total fillable pore space can be made by means of a gas porosimeter than from determinations of absorption by the boiling test. It may be, therefore, that the failure to find a satisfactory correlation between saturation coefficient of cast concrete and in particular cast stone and resistance to frost action resulted from the unreliable method for determining values of the coefficient.[2]

As a large proportion of concrete block are used in portions of structures which may be made unsightly if shrinkage is excessive, users of these products are much concerned with information on volume change. Insofar as comparisons are available, the data presented by the authors agree qualitatively with those previously published by Allan.[3] However, the values reported by Allan were considerably larger. It would be of interest if the authors could present an explanation for this difference and would state their recommendations as to the most reliable way for the purchasers of block to estimate the probable shrinkage of the units after laying. Current specifications place a limit on the amount of moisture in the units at time of delivery, measured in terms of the total absorption of the units. If the authors could suggest a better basis it would be worthwhile for them to do so.

AUTHORS' CLOSURE

In answer to Mr. Shields and Mr. Wilk, the authors did not intend that a specification requirement of 1,000 p.s.i. at 28 days should apply to all concrete masonry units regardless of where they are used in the structure. It was only intended as a limitation for those units used in structures where severe exposure might occur, such as in the north eastern quarter of the United States. This is specifically provided for in Conclusion 12. For interior or backup construction the current minimum strengths of the American Society for Testing Materials specifications are satisfactory.

In answer to Mr. Wilk's question regarding satisfactory strength requirements at ages earlier than 28 days; in Conclusion 11 where strength and rate of absorption were used as the criteria of satisfactory durability, substituting a 700 p.s.i. at 7 days for the 900 p.s.i. at 28 days would have caused no change in the effectiveness of the

[1]Tentative Specifications for Building Brick Made from Clay or Shale (C 62-37) *Proceedings*, A. S. T. M. Vol. 37, part I, page 733, 1937.
[2]The Physical Properties of Cast Stone, JOURNAL, Amer. Concrete Inst. Dec. 1931; *Proceedings* Vol. 28, page 243.
[3]Shrinkage Measurements of Concrete Block Masonry (Final Report) JOURNAL Amer. Concrete Inst., Nov. 1931; *Proceedings* Vol. 28, page 177, 1932.

conclusion. Similarly in Conclusion 12 where strength alone was the criteria, substituting 800 p.s.i. at 7 days for 1,000 p.s.i. at 28 days would not have changed the effectiveness of the conclusion.

In answer to Mr. Parsons, the authors fully realize the limitation of this investigation in so far as effectiveness in variation of grading is concerned. It is desirable that any further studies should expand upon this variable. Some studies of this character are now under way and it is hoped that further information may be presented.

With reference to the difference in the extent of volume change reported by Allan and in this paper for concrete masonry, undoubtedly a considerable part of this difference is due to the fact that in Allan's paper measurements were made of 8 units and 7 mortar joints and in this paper the measurements were made within a single unit.

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *NOVEMBER 1939*

The Corridor Beam Floor*

By C. A. Willson†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

This paper describes a shallow floor system which was devised for the new men's dormitories at the University of Wisconsin. Since beams of the usual type were eliminated, it was possible to reduce story heights, cut the cost of form work, and simplify the erection of partitions. By varying the details of slab thickness and amount of reinforcement, the fundamental ideas presented here could be utilized to advantage in a variety of projects.

At the University of Wisconsin, Madison, a group of eight new men's dormitories capable of housing a total of 632 students has just been completed at a total cost of $600,000. Each of these buildings is 36 ft. wide, 112 ft. long and has three dormitory stories and a basement. The project was financed partially by state funds and partially by PWA grants.

To be in harmony with other buildings on the campus it was decided to use reinforced concrete floors and roof and a stone exterior facing. For the sake of economy it was agreed that the story heights should be reduced to a minimum and that the concrete surfaces without plaster should constitute the ceilings.

The problem was to secure a very shallow floor system which would fit the arrangement of rooms and give a pleasing ceiling appearance. After a number of preliminary designs were made and studied the scheme illustrated in Fig. 1 was adopted. The floor slab proper, marked Slab A, might be considered as a three-span continuous structure consisting of two long spans and one short span, or as two slabs continuous over a wide central supporting member. This central member, marked Corridor Beam B, is the only visible beam on each floor and

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†Structural Engineer, State Architect's Office, Madison, Wisc.

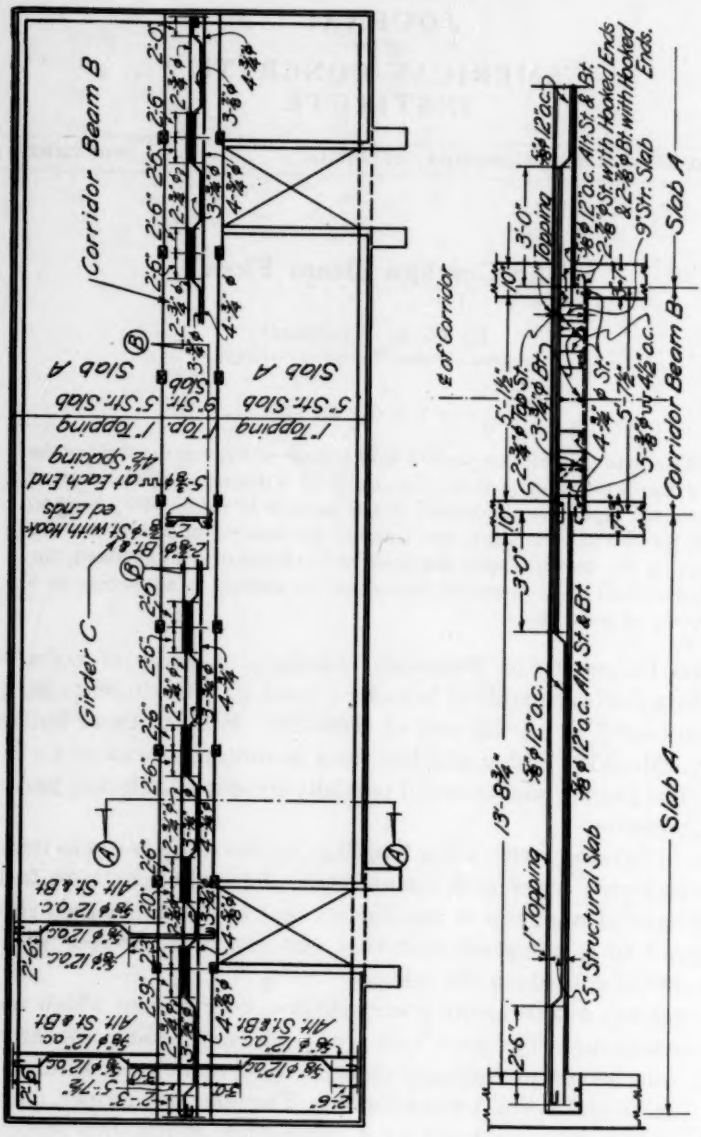

FIG. 1 (TOP)—TYPICAL FLOOR PLAN
FIG. 2 (BOTTOM)—SECTION A-A

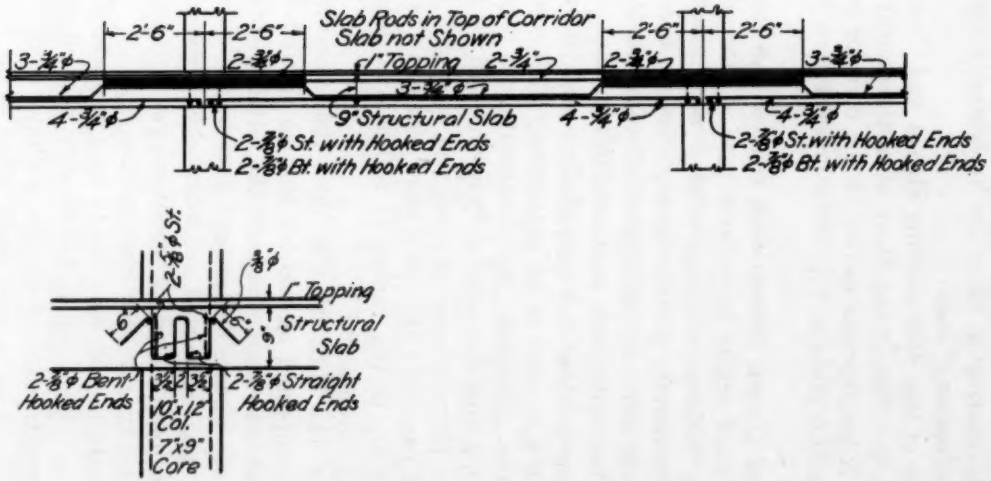

FIG. 3 (TOP)—SECTION B-B (SEE FIG. 1)
FIG. 4 (BOTTOM)—DETAIL OF GIRDER C

is actually a continuous beam extending the length of the building. A short beam, marked Girder C, supports the Corridor Beam B at intervals of 12 ft. 6 in. While these two members are designed as though they were separate and distinct, yet they have the same depth and therefore can be formed in the simplest manner. From the standpoint of the contractor, there are no beams in the whole floor, but just two thicknesses of slabs.

A cross section of this floor showing Slab A and Corridor Beam B is shown in Fig. 2. The arrangement of reinforcement in Girder C is shown also. A longitudinal section of the corridor beam showing bar bending details is given in Fig. 3. A cross section of Girder C is indicated in Fig. 4.

On the basis of normal unit stresses, smaller slab thicknesses and less reinforcing steel might have been used. However, the small increases in slab thickness and area of steel were adopted to reduce plastic flow to a minimum. Furthermore, vibrators were used during construction to cut down on the amount of water needed for placing the concrete. The results were very satisfactory.

It must be conceded that in a wide shallow corridor beam the reinforcing steel cannot be used as advantageously as it would be in two normal deep beams. However, this disadvantage is more than offset by savings resulting from decreased story heights and the simplification in construction of forms, placing of steel, and placing of concrete. In addition, tops of partitions are fitted more readily to two slab thicknesses than they are to the variety of widths and depths of beams which are usually encountered.

The contractor who built the first three dormitory buildings used plywood panels supported on wooden shoring to form the two slab thicknesses. The contractor who built the last five dormitory buildings used a smooth surface compressed fibre board laid on a wooden sub-floor which was supported on adjustable steel joists. While these two contractors differed in procedure, yet they agreed in their enthusiasm for this type of floor construction.

It is not claimed that this idea can be used economically under any and all conditions. However, it has already been incorporated into the design and construction of another building of considerable consequence and is likely to be used many times in the future as knowledge of its advantages becomes more widespread. This type of floor should be considered wherever low-cost fireproof construction is contemplated.

Discussion, to close in April, 1940 JOURNAL, *should reach A. C. I. Secretary in triplicate by Feb. 1, 1940.*

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *APRIL 1940*

Discussion of a paper by C. A. Willson

The Corridor Beam Floor*

BY H. M. HADLEY†

It is an admirable floor that Mr. Willson describes. There is much more to the economical design of reinforced concrete than the mere quantities of concrete and reinforcement involved. There is the shape and cross-section of members, the matter of getting concrete into place and around reinforcement, the cost of plastering which is consequent on respective designs where plastering is used, the story heights, the reactions set up in contractors when they examine plans and make up their bids. It is not at all surprising to read that both of Mr. Willson's contractors were enthusiastic for this floor. Of course they were. It looked easy to build and they found it easy to build and when the construction was completed they had created something that they could take genuine satisfaction in: a thoroughly good piece of work.

It would be a most wholesome salutary review to apply to every engineering design for its designer seriously, soberly to ask himself the question, "If I had to build this thing myself, how would I do it?" This might be supplemented by the question, "If I had to pay for it myself, how would I like it?" There would be a marked reduction of the "God awful" type of structures could such self-questioning be made mandatory. Mr. Willson's floor would withstand a searching, rigorous test of this nature. It is true some additional material was put in for the sake of "plastic flow"—which at most would have been of a put-in-your-eye magnitude—but the virtues and economies of this floor are so pronounced that only serenity and

*JOURNAL Amer. Concrete Inst., Nov., 1939; *Proceedings* Vol. 36, p. 165.
†Regional Structural Engineer, Portland Cement Association, Seattle, Wash.

peace of mind could follow on its most impersonal and critical examination.

The synopsis of Mr. Willson's article speaks of this floor as being "devised" for these dormitories. This should not be interpreted as indicating its original use in them. The *Engineering News-Record* of Jan. 9, 1930, p. 68 published an article by George Runciman and Harold Worthington of Seattle entitled "One-Way Slab Floors Being Used Extensively in the Northwest." Therein is illustrated and shown in detail this same corridor beam-slab floor used in the Rhododendron Apartments in Seattle on a somewhat larger scale, the heavy central slab being extended into the exterior bays and completely surrounding the columns. Incidentally it may be observed that the Olympic Hotel floors, referred to in the article's tabulation as being 5 in., 5½ in., 5 in. thick on clear spans of 15 ft. 2 in., 18 ft. 7 in., 14 ft. 2 in. and built in 1923 have never plastically flowed enough to reveal that fact by diagonal hair cracks in the plaster of the partitions or by other appreciable evidence. Perhaps, called "shrinkage," or "drying out," it had occurred before partitions and plaster came onto the job.

It is interesting to note that in this University of Wisconsin work at a total cost of $600,000 housing accommodations are provided for 632 students. This is an investment of $950 per student. It would also be interesting to know how much of this per capita investment abides in the stone exteriors of these buildings. "Stone walls do not a prison make, nor iron bars a cage" but sometimes stone walls do rather seriously curtail the interior dimensions of rooms and corridors. The writer has in mind a Pacific Coast institution of limited means one of whose principal buildings was provided with a stone exterior, authentic of the times of Edward the Confessor, or whom you will. The result was magnificent stone work, miserably small ill-lighted classrooms, and the implications, gently ironic, of mediaeval or classical architecture in a setting of Douglas firs. Another effect which may be called an implication was that several subsequent lesser buildings were erected of wood frame and stucco. *"Sic transit gloria mundi."*

AUTHOR'S CLOSURE

Mr. Hadley seems inclined to smile at the author's concern in regard to plastic flow. However, serious cracks in plaster partitions have been caused by the gradual yielding of a very shallow floor over a period of years and the author was determined to guard against such an occurrence. With our increases in allowable unit stresses more attention will need to be given to this subject in the future.

In their article in the *Engineering News-Record,* it seems most unfortunate that Messrs. Runciman and Worthington did not feature the design of the floor used in the Rhododendron Apartments for it appears to be quite similar to the floor under discussion. The twenty other buildings listed in their article involve nothing out of the ordinary, but they seem to be well described. The earlier use of the corridor beam floor apparently was not generally known for Mr. A. J. Boase, Manager of the Structural Bureau of the Portland Cement Association came to Madison to see the buildings and the complete design drawings as soon as he saw the author's paper.

Mr. Hadley raises natural questions about the investment per student and the use of stone walls. It should be explained that the total cost of the project includes complete dining rooms, kitchens, offices, social rooms, recreation rooms, etc. and therefore the cost per student is out of balance. Stone walls were used because most of the buildings at the University have been built of that material—a local product which is pleasing in color and texture.

In conclusion, the author wishes to thank Mr. Hadley for the time and effort he has given to the preparation of his discussion.

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *NOVEMBER 1939*

Tests of the Resistance to Rain Penetration of Walls Built of Masonry and Concrete*

By R. E. Copeland†
MEMBER AMERICAN CONCRETE INSTITUTE

and C. C. Carlson†

SYNOPSIS

The investigation reported in this paper pertains to the problem of building rain-proof masonry walls. Masonry wallettes, 32 in. wide, 48 in. high and 4 in., 8 in. and 12 in thick were exposed to a simulated wind-driven rain and the rate of moisture penetration was determined. Construction types included cast-in-place concrete and unit masonry of brick, concrete block and tile and combinations thereof. Other factors such as quality of workmanship and mortar composition were studied. Considerable information was obtained concerning the effectiveness of cement paints as a water resistant coating on concrete masonry. The performance of the brick walls was influenced principally by quality of workmanship. The plasticity of the lime used in the mortar also was a significant factor in the brick wall tests but mortar composition as regards the ratio of cement to lime content was unimportant. Results are summarized in 33 conclusions.

Since publication of the progress report in 1936‡ permeability tests have been performed on 101 wall specimens built of concrete masonry, cast-in-place concrete and solid brick faced masonry and the investigation has been concluded.

This paper is a summary of the entire study embracing over 200 permeability tests conducted for the purpose of developing information on the resistance of various wall constructions to wind driven rain and on various factors which presumably importantly influence the permeability of masonry.

*Received by the Institute Aug. 8, 1939
†Development Department, Portland Cement Association.
‡"Tests of the Resistance of Concrete Masonry Walls to the Penetration of Rain," by R. E. Copeland and C. C. Carlson; JOURNAL of the Amer. Concrete Inst., Mar.-Apr., 1936. *Proceedings*, Vol. 32, p. 485.

SCOPE OF INVESTIGATION

Considering the concrete masonry walls, the factors studied included: type and grading of aggregate, concrete mixtures and curing conditions, wall thickness, mortar composition, joint treatment, surface texture, kind and application of different surface waterproofings, and the effect of weather exposure on paint coatings.

The brick wall tests pertained to the effects of type and properties of the facing and back-up units, mortar composition, quality of workmanship, construction details and wall thickness.

Various supplementary tests and a few walls of reinforced concrete were included.

The scope of each of the individual series was as follows:

Series A

Twenty-seven plain concrete masonry walls built of 8 x 8 x 16 in., 3 oval core block made of limestone, cinder, Haydite, and sand and gravel concrete were tested to study the effect of type and grading of aggregate, concrete mixtures and exposed surface texture. Representative walls were retested after receiving one or two coats of portland cement paint. Five plain walls were retested after painting over the joints with portland cement paint. One cinder block wall with three coats (about 1 in.) portland cement stucco was included.

Series B

Six concrete masonry walls built of 8 x 8 x 16 in., 3 oval core Haydite block were first tested plain to determine the effect of different mortar composition and joint construction. Two walls spray painted with portland cement paint and two walls brush painted were retested to show the relative efficiency of these two methods of paint application.

Series C

Twelve 4-in. thick concrete masonry walls built of 4 x 8 x 16 in., 3 core partition units made of Haydite concrete and sand and gravel concrete were tested with plain and painted surfaces. Two gradings of aggregate and three mix proportions were included for each type of aggregate.

Series D

Fifteen concrete masonry walls built of 8 x 8 x 16 in., 3 oval core cinder block were painted with 8 job mixed cement base compositions and 6 representative proprietary cement paints and tested to determine the waterproofing effectiveness of a wide range of cement base coatings.

Series E

Following their initial test, representative painted walls from Series A and D were set outdoors with their painted surfaces exposed to the weather. These walls were retested one to three times during a weather exposure period of from 8 to 32 months.

Series F, F1 and F2

Thirty-six brick walls of 4-in. face brick and 4-in. common brick back-up, nine 8-in. common brick walls and nine 8-in. select common brick walls were tested. There were two grades of workmanship, four mortar compositions ranging from high cement to straight lime types with and without water-proofing and three forms of lime as used giving a substantial difference in mortar plasticity and moisture retentivity.

Series F3

Fourteen 12-in. walls of 4-in. face brick with 8-in. common brick and cinder concrete block back-up and five 8-in. walls of 2-in. face brick with 6-in. cinder and limestone concrete masonry back-up were tested. There were two grades of workmanship. In addition to the influence of this factor, information was obtained on the comparative efficiencies of the different types of back-up construction and on the previousness of walls built with the 2-in. thick (split) brick facing.

Series G

Eight walls of plain and painted concrete masonry, brick masonry and cast-in-place concrete were subjected to four different combinations of wind and rain intensities ranging from a minimum of ½-in. rain intensity with a 25 m.p.h. wind velocity to a maximum of a 2½-in. rain intensity with a 56 m.p.h. wind velocity. These combinations were intended to cover a wide range in rainstorm intensity and to provide comparative results having possible value in broadening the application of the investigation as a whole.

Series H

Eight walls were tested to study the waterproofing effectiveness of four different commercial compounds used on cinder and limestone block masonry. One compound was a heavy-bodied tung oil paint; the other three were thin-bodied liquids of the colorless type.

Series I

In this series further study was given to the durability of paint coatings on concrete masonry. Walls were built of commercial masonry units obtained from various localities. Four different cement base paints and an equal number of oil paints were applied to each wall

TABLE 1—DETAILS OF AGGREGATES, MIXES AND PHYSICAL PROPERTIES OF CONCRETE MASONRY UNITS; SERIES A, B, C, D, J, AND H.

Wall Code No.	Aggregate			Concrete			
	Type	Max. Size; Inches	Fineness Modulus	Mix, By Dry Rodded Aggregate Volume	Comp. Strength on Gross Area; p.s.i. (1)	Absorption 24 Hr. Immersion	
						% of Dry Weight	% of Volume
Series A and B Walls; 8 by 8 by 16 inch 3 oval core, 32.3% Core Space Block							
AC-1	Cinders	⅜	4.0	1–8	780	14.7	20.9
AC-2	"	"	3.5	1–6	1270	14.1	20.7
AC-3	"	"	3.5	1–8	820	15.2	21.9
AC-4	"	"	3.5	1–10	510	16.9	23.7
AC-5	"	⅝	4.5	1–6	1420	12.1	17.6
AC-6	"	"	4.5	1–8	1070	12.5	17.6
AC-7	"	"	4.5	1–10	870	12.9	17.6
AC-8 (2)	"	¾	3.5	1–8	700	14.9	21.6
AC-9 (2)	"	⅝	4.5	1–8	910	12.9	18.3
AH-1	Haydite	⅜	3.5	1–7	1390	18.2	20.6
AH-2	"	"	3.5	1–9	1120	18.8	20.7
AH-3	"	"	3.5	1–11	1100	20.6	22.7
AH-4	"	⅝	4.5	1–7	1220	15.6	17.2
AH-5	"	"	4.5	1–9	1370	15.8	17.6
AH-6	"	"	4.5	1–11	1110	15.5	15.8
BH-7	"	¾	4.0	1–9	1150	16.9	18.1
AH-8 (2)	"	"	3.5	1–9	1240	17.8	20.1
AH-9 (2)	"	⅝	4.5	1–9	1030	14.7	16.3
AS-1 (3)	Sand & Gravel	⅜	3.5	1–8	1770	7.7	15.6
AS-2 (3)	"	"	3.5	1–10	1320	7.6	15.2
AS-3 (3)	"	⅝	4.5	1–8	2310	5.8	12.5
AS-4 (3)	"	"	4.5	1–10	1820	5.7	12.2
AL-1	Limestone	⅜	3.5	1–8	1380	7.7	16.2
AL-2	"	"	3.5	1–10	1330	8.1	16.7
AL-3	"	⅝	4.5	1–8	1540	7.2	15.3
AL-4	"	"	4.5	1–10	1270	7.5	15.4
Series C Walls; 4 by 8 by 16 inch 3 core, 24.3% Core Space Tile							
CH-1	Haydite	⅜	3.5	1–7	1050	17.9	19.6
CH-2	"	"	3.5	1–9	890	19.9	20.8
CH-3	"	"	3.5	1–11	830	21.5	22.1
CH-4	"	"	4.25	1–7	900	15.6	16.0
CH-5	"	"	4.25	1–9	900	15.5	14.9
CH-6	"	"	4.25	1–11	730	17.5	16.6
CS-1	Sand & Gravel	"	3.5	1–6	2250	7.7	15.0
CS-2	"	"	3.5	1–8	1940	7.6	14.9
CS-3	"	"	3.5	1–10	1140	8.2	15.5
CS-4	"	"	4.25	1–6	2770	5.8	12.0
CS-5	"	"	4.25	1–8	2080	6.9	13.8
CS-6	"	"	4.25	1–10	1690	7.2	14.3
Series D, J and H Walls; 8 by 8 by 16 inch 3 oval core, 36.5% Core Space Block							
D-1 to 15	Cinders	½	4.0	1–8	704	14.2	21.0
J-1 (4)	Limestone	⅜	3.75	1–10	1810	8.5	18.8
J-2	Haydite	½	3.75	1–9	1740	15.1	21.0
H-1,3,5,7 (5)	Cinders	—	——	——	940	15.3	20.9
H-2,4,6,8	Limestone	⅜	3.90	1–10	1060	8.3	16.3

NOTES:

(1) All specimens air dry when tested. Age of specimens at test: Series A, B, C, J and H, 3 to 6 months; Series D, 28 days.

(2) Oscillated face block—40% core space.

(3) Water eroded face block used in walls AS-5, AS-6, AS-7 and AS-8 of same composition as AS-1, AS-2, AS-3, and AS-4 respectively.

(4) Pulverized silica substituted for 30 per cent of the cement.

(5) Purchased from commercial stocks.

Curing: J Series block cured in high pressure steam (350°F.), all others cured 24 hours in steam vapor followed by air storage except AC-8, AC-9, AH-8, and AH-9, which had 3 days damp curing followed by storage in air.

TABLE 2—DETAILS OF AGGREGATES, MIXES AND PHYSICAL PROPERTIES OF CONCRETE OF SERIES G REINFORCED CONCRETE WALLS

Walls were 4″ thick, 48″ high, 32″ long.
Walls G-"A" and G-"B" cast vertically in plywood forms. Wall G-"C", consisting of a 3″ layer cinder concrete faced with 1″ layer sand and gravel concrete, was cast flat.
Walls were cast in continuous operation—no construction joints. Concrete was hand spaded.

Wall Code No.	Aggregate				Concrete					
	Type	Max. Size; Inches	Fineness Modulus	Dry Rodded Weight; Lb./C.F.	Mix—by Dry Rodded Aggregate Volume	Total Water(1) Per Sack Cement; Gallons	Slump; Inches;	Comp. Strength; p.s.i.(2)	Absorption 24 Hr. Immersion	
									% of Dry Wgt.	% of Volume
G-"A"	Sand	¼	3.3	116.5	1-2.1-3.1	8.7	5½	3660	6.6	14.9
	Limestone	¾	6.7	94.5						
G-"B"	Sand	¼	3.3	116.5	1-1.7-3.6	7.5	5½	4270	5.0	11.7
	Gravel	¾	6.9	108.0						
G-"C"	Cinders	⅝	4.2	79.6	1-4.75	13.0	2½	1470	13.5	23.7
	Sand	¼	3.3	116.5	1-1.7-0.92	5.9	9	6890	5.2	11.7
	Gravel	½	6.4	103.8						

NOTES: (1) Includes moisture (free and contained) in aggregate.
(2) Tests made on 3 by 6 inch cylinders, 28 days old and in air dry condition.

TABLE 3—PHYSICAL PROPERTIES OF MASONRY UNITS IN BRICK WALLS; SERIES F, F1, F2 AND F3

BRICK

Series	Wall No.	Description of Unit	Modulus of Rupture; p.s.i.	Compressive Strength; p.s.i.	Absorption			
					24 Hr. Immersion		5 Hr. Boil (B) % of Dry Wgt.	$\frac{C}{B}$ Ratio
					% of Dry Wgt. (C)	% of Volume		
F	1 to 12 incl.	4″ Face	1,550	13,330	5.1	10.8	8.5	0.60
F	13, 15, 17	4″ Select Common	1,110	4,910	8.9	16.0	14.1	0.66
F	1 to 12 & 14, 16, 18	4″ Common	578	3,500	21.5	34.7	25.6	0.84
F1 & F2	1 to 12 incl.	4″ Face	1,860	14,300	6.5	13.5	9.5	0.68
F1 & F2	13, 15, 17	4″ Select Common	1,220	3,540	11.3	20.0	16.9	0.67
F1 & F2	1 to 12 & 14, 16, 18	4″ Common	690	3,760	20.2	32.4	24.2	0.83
F3	1, 2, 3 & 8 to 19	4″ Face	1,860	14,300	6.5	13.5	9.5	0.68
F3	1	4″ Common	690	3,760	20.2	32.4	24.2	0.83
F3	4, 5, 6, 7	2″ Face	2,640	17,300	1.7	4.1	5.65	0.31
F3	4, 5, 6, 7	4″ Face (Headers)	1,760	18,700	3.6	8.2	5.57	0.65

CONCRETE MASONRY BACK-UP UNITS (1)

Series	Wall No.	Description of Unit	Compressive Strength; p.s.i. of Gross Area	Absorption 24 Hr. Immersion	
				% of Dry Wgt.	% of Volume
F3	2 & 12 to 19	8 by 8 by 16 inch—3 oval core cinder concrete block	940	15.3	20.9
F3	3	4 by 8 by 16 inch—3 rect. core cinder concrete tile	1,100	12.9	17.9
F3	4, 5	6 by 8 by 16 inch—3 rect. core cinder concrete units	1,120	13.1	20.0
F3	6, 7	6 by 8 by 16 inch—3 rect. core limestone conc. units	1,650	7.7	16.0

NOTE: (1) Purchased from commercial stocks.

and subjected to weather exposure. Volume change of the masonry and the condition of the paint are being kept under observation.

Series J

One wall each was built of limestone and Haydite concrete block cured with high pressure steam. These walls were first tested plain and then retested after being painted.

Supplementary Studies

In addition to the above series a number of tests were made to determine the effect on rain penetration of hermetically sealing the perimeter of the test wall as there was some question if such treatment would affect the results or was necessary to obtain a specimen condition similar to a small section within a large building wall area.

Throughout the investigation various check tests were made to verify initial results and particularly the constancy of the test method.

PREPARATIONS FOR TEST PROGRAM

Manufacture of Concrete Masonry Units

Manufacturing data and physical properties pertaining to the concrete units are given in Tables 1 and 3.

The cinder concrete units for series H and F3 walls were purchased from commercial stocks. The concrete masonry units for all other series were made in power tamper, stripper block machines specifically for the test program.

The oscillated face cinder and Haydite concrete units employed for four walls in series A were made in a stripper machine having a special attachment which produced a trowelling action on the face of the unit simultaneously with the tamping.

The water eroded sand and gravel concrete units employed for four walls in series A were made in the same manner as plain face units but before the concrete had attained its initial set, the block face to be test exposed was water eroded with a fine garden hose spray. This treatment washed out the cement and fine sand to a depth of about ¼-in., thus revealing the coarse aggregate and imparting a rough texture.

Brick Units

The physical properties of the brick are given in Table 3.

The face brick used in Series F, F1 and F2 and for the 12 -in. walls of Series F3 were of mat texture made of shale by the stiff mud, side cut process.

The 2-in. split brick and standard size face brick used in walls F3-4, 5, 6 and 7 were a mat texture, side cut, red shale brick.

The select common and common brick were made of surface clay by the stiff mud, end cut process. The select grade was somewhat better burned and truer in shape than the regular common grade.

Mortar Materials and Cement

All cement used was a mixture of equal parts of four brands of normal portland cement purchased from dealers.

The lime was purchased from commercial stocks and was of the high calcium type. The lime constituent in all mortars was either a dry or presoaked hydrated lime except that in Series F2 a putty made with pulverized quicklime was employed.

The mortar sand gradings are tabulated below:

MORTAR SAND GRADINGS

Screen Size	Per Cent Retained on Screen		
	Series (A, B, C, D)	Series F	Series (F1, F2, F3, H, J)
No. 4	0	0	0
No. 8	0.20	0.5	0
No. 14	16.60	5.5	4.6
No. 28	41.95	28.5	22.9
No. 48	85.20	84.5	82.0
No. 100	97.55	98.4	98.3
Fineness Modulus	2.42	2.17	2.08

The coarser sand was satisfactory for the mortars employed in laying up concrete block. In the brick wall series it was found advisable after some experimentation to use the finer grading as indicated.

Erection of Concrete Masonry Walls

These walls were built 6 courses (about 48 inches) high by 2 block lengths (about 32 inches) long. Mortar was a 1:1:6 cement-lime mix by volume based on dry rodded sand for series A, B, C and D and damp loose sand for series H and J. The hydrated lime was mixed dry with the cement and sand before gaging.

The walls were carefully constructed by the same mason in an effort to minimize differences in workmanship. Joints were $\frac{3}{8}$-in. thick and were tooled concave. Two walls in Series B were laid with full mortar bedding. All other walls were laid with face shell mortar bedding, that is, no mortar was placed on the transverse webs. All units were air dry, not wetted, when used.

Erection of Brick Walls

Table 4 gives the physical properties of the different mortar compositions employed in laying the brick walls in series F, F1, F2 and F3.

Fig. 1, 2 and 3 and 5 to 8 illustrate the wall construction details. They were approximately 33 in. wide by 50 in. high. Mortar joints were $\frac{3}{8}$-in. thick in the face brick walls and $\frac{1}{2}$-in. thick in the common

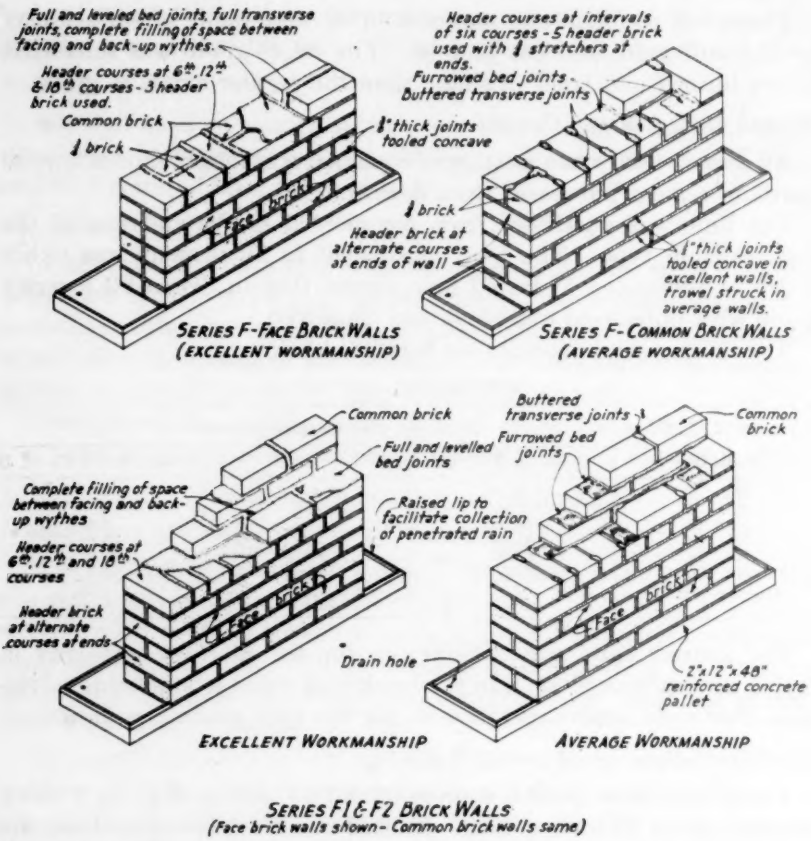

Full and leveled bed joints, full transverse joints, complete filling of space between facing and back-up wythes.
Header courses at 6th, 12th & 18th courses - 3 header brick used.
Common brick
½ brick
Face brick
½" thick joints tooled concave
SERIES F-FACE BRICK WALLS
(EXCELLENT WORKMANSHIP)
Header courses at intervals of six courses - 5 header brick used with ½ stretchers at ends.
Furrowed bed joints - Buttered transverse joints
½ brick
Header brick at alternate courses at ends of wall
½" thick joints tooled concave in excellent walls, trowel struck in average walls.
SERIES F-COMMON BRICK WALLS
(AVERAGE WORKMANSHIP)
Common brick
Full and levelled bed joints
Complete filling of space between facing and back-up wythes
Header courses at 6th, 12th and 18th courses
Header brick at alternate courses at ends
Face brick
EXCELLENT WORKMANSHIP
Buttered transverse joints
Furrowed bed joints
Common brick
Raised lip to facilitate collection of penetrated rain
Face brick
Drain hole
2"x12"x48" reinforced concrete pallet
AVERAGE WORKMANSHIP
SERIES F1 & F2 BRICK WALLS
(Face brick walls shown - Common brick walls same)

FIG. 1—CONSTRUCTION DETAILS OF SERIES F, F1 AND F2 8-IN. BRICK WALLS

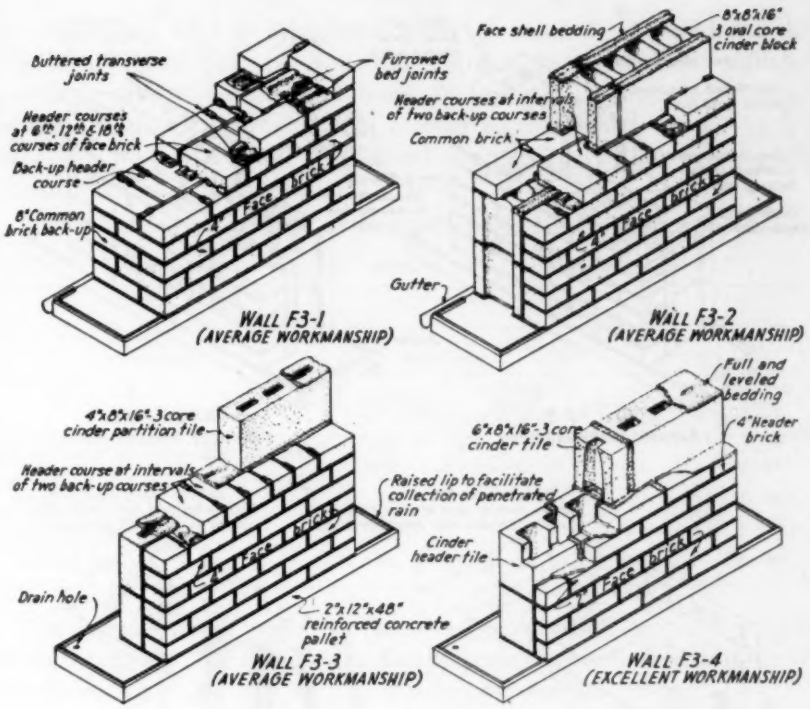
Face brick walls built 19 courses high; common 17 courses high. All walls approx. 33" wide. Joints of face brick walls ⅜" thick-tooled concave. Joints of common brick walls ½" thick-tooled concave in case of excellent workmanship and struck with point of trowel in case of average workmanship.

Excellent workmanship characterized by full and carefully leveled bed joints, shoved and full transverse joints, complete filling of space between facing and back-up wythes and careful tooling of exposed face joints after mortar had started to stiffen.

Average workmanship characterized by furrowed bed joints, buttering of transverse joints near faces of wall, partial filling of space between facing and back-up wythes and average tooling of joints as noted.

and select common brick walls. In constructing the walls the facing wythe was erected slightly ahead of the back-up units except that for the walls having 2 in. (split) brick facing it was necessary to reverse the order because of the tendency of the thin facing to slide and tilt out of line when the back-up was placed afterward.

Series F1, F2 and F3 walls were built by the same mason, a bricklayer by trade for about 20 years. A different mason, equally experienced, erected series F walls.



NOTE: *Excellent workmanship* characterized by full and carefully leveled bed joints, shoved and full transverse joints for both brick and concrete masonry units, complete filling of space between facing and back-up wythes and careful tooling of exposed face joints after mortar had started to stiffen.

Average workmanship characterized by furrowed bed joints for brick courses and face shell bedding for concrete masonry courses, buttering of transverse joints near faces of wall, partial filling of space between facing and back-up wythes and average tooling of exposed face joints.

Walls built approx. 32″ wide, 48″ to 50″ high.

Joints of face brick ⅜″ thick—tooled concave.

Construction details of wall F3-5 same as that of F3-4 except for use of "average workmanship." Wall F3-6 same as F3-7 except for use of "excellent workmanship."

FIG. 2—CONSTRUCTION DETAILS OF SERIES F3 WALLS

There were two grades of workmanship. Excellent workmanship was characterized by full and carefully levelled bed joints, full and shoved head joints, solid filling of the longitudinal joint between wythes and careful concave tooling of the joints after the mortar

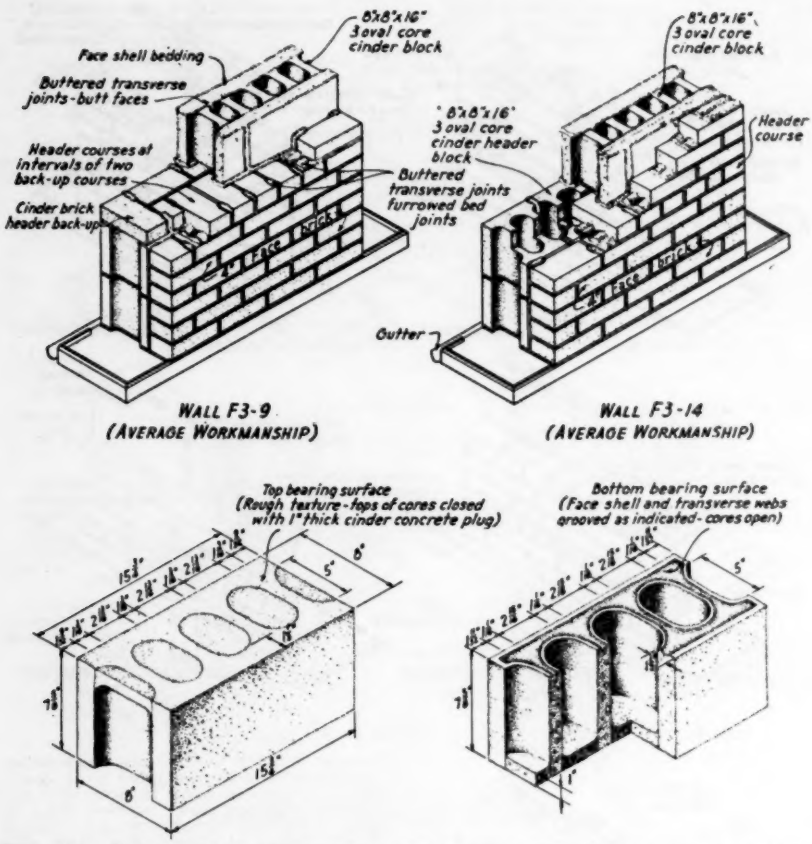

8"x8"x16" 3 oval core cinder block
Face shell bedding
Buttered transverse joints-butt faces
Header courses at intervals of two back-up courses
Cinder brick header back-up
Buttered transverse joints furrowed bed joints
4" Face Brick
WALL F3-9
(AVERAGE WORKMANSHIP)
8"x8"x16" 3 oval core cinder block
8"x8"x16" 3 oval core cinder header block
Header course
Gutter
WALL F3-14
(AVERAGE WORKMANSHIP)
Top bearing surface
(Rough texture-tops of cores closed with 1" thick cinder concrete plug)
Bottom bearing surface
(Face shell and transverse webs grooved as indicated-cores open)

FIG. 3—CONSTRUCTION DETAILS OF SERIES F3 WALLS—CINDER CONCRETE UNITS USED AS BACK-UP IN WALLS F3-16, 17 AND 19. CORE AREA—39 PER CENT

(Similar block with open cores in walls F3-9, 12, 12E-18)

NOTE: Refer to Fig. 2 for description of Excellent and Average Workmanship.

had started to stiffen perceptibly. Brick previously examined and passed as free from bad cracks, chipped edges and warpage were used for the walls of excellent workmanship.

Average workmanship was characterized by use of run of average brick, furrowed bed joints, buttered head joints, incomplete filling of the joints between wythes and rather less careful tooling of the joints. The joints of the common brick walls of average construction were smoothed flush with the end of the trowel.

The photographs (Fig. 5 to 8) taken during the erection of the brick walls clearly show the differences between "excellent" and "average" workmanship.

Considering the 36 face brick walls of series F, F1 and F2 the average building time per wall was 2 hours, 48 minutes for walls of excellent workmanship and 1 hour, 57 minutes for walls of average workmanship. The quantity of mortar used varied from 1.7 to 2.3 cu. ft. for walls of average workmanship and 2.1 to 2.4 cu. ft. for walls of excellent workmanship.

These data for series F3 walls are given in the following table:

MORTAR QUANTITIES AND BUILDING TIME, SERIES F3 WALLS

Wall Description	Workmanship			
	Excellent		Average	
	Mortar Used; Cu. Ft.	Building Time Hr.—Min.	Mortar Used; Cu. Ft.	Building Time Hr.—Min.
12″ walls of 4″ face brick & 8″ common brick back-up.	3.7	3—48	3.3	2—55
12″ walls of 4″ face brick & 8″ cinder block back-up.	2.6	2—43	2.0	1—58
8″ walls of 4″ face brick & 4″ cinder tile back-up.			1.6	1—46
8″ walls of 2″ face brick & 6″ concrete masonry back-up.	1.6	2—25	1.4	2—30

Neither the face brick nor concrete block back-up were wetted prior to laying. The common and select common brick were thoroughly sprinkled with a garden hose to prevent excessive suction.

Mortar batches were mixed for one wall at a time. For series F the mason was permitted to retemper the mortar on the board to suit his requirements. This was not entirely satisfactory and for series F1, F2 and F3 the steel mortar box was fitted with a tight cover to prevent excessive evaporation and no water was added to the mortar after the initial gaging to a consistency approved by the mason.

In series F1, the hydrated lime was thoroughly mixed with an equal weight of water to form a thick creamy paste which was stored in a tightly covered can for a 24-hr. presoaking period before being used.

The high calcium fast slaking pulverized quicklime used in series F2 was slaked 72 hours in advance employing approximately 106 lb. of water in slaking 50 lb. of lime.

Erection of Reinforced Concrete Walls

Three 4-in. thick reinforced concrete walls were built for tests under Series G. The construction details are given in Table 2. Three sets of internal electrodes, consisting of pairs of $\frac{3}{8}$ in. round, smooth steel rods spaced $\frac{5}{8}$ in. apart were cast in each of the walls $1\frac{9}{16}$ in. back from the exposed face to serve the dual purpose of reinforcement and of facilitating the detection of internal rain penetration.

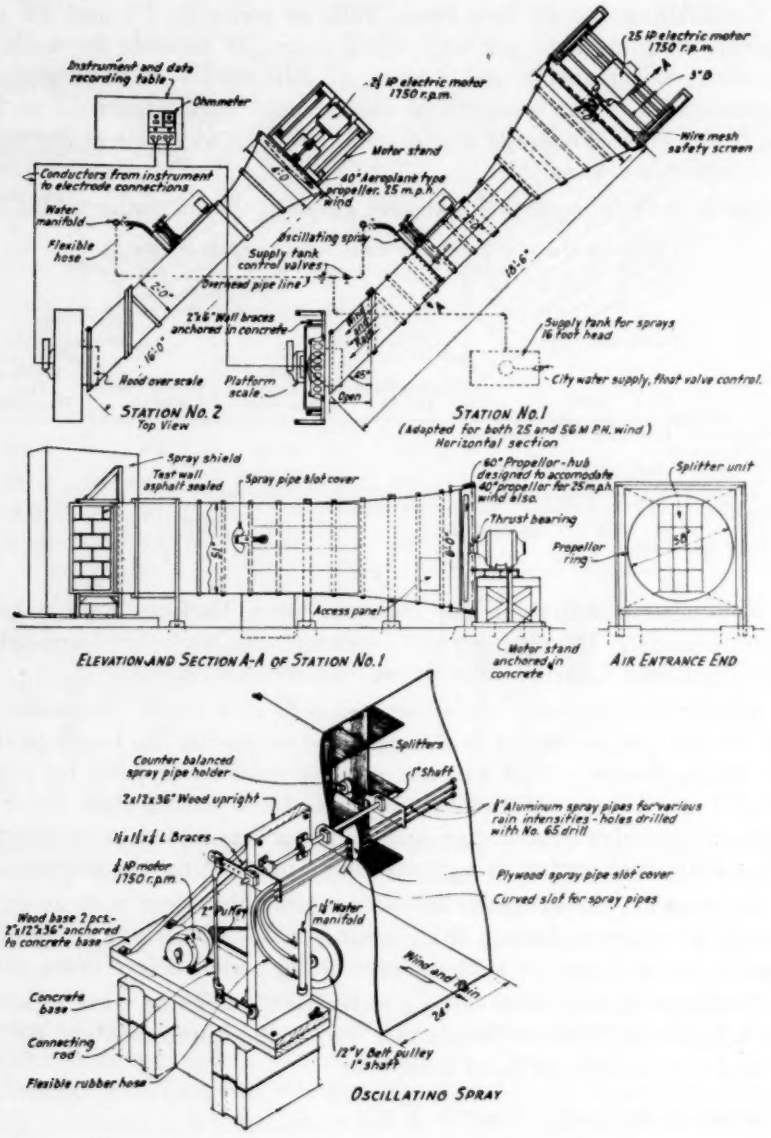

Instrument and data recording table
Ohmmeter
Conductors from instrument to electrode connections
Water manifold
Flexible hose
2½ HP electric motor 1750 r.p.m.
Motor stand
40° Aeroplane type propeller, 25 m.p.h. wind
Oscillating spray
Supply tank control valves
Buried pipe line
2'x6" Wall braces anchored in concrete
Hood over scale
Platform scale
STATION No. 2
Top View
25 HP electric motor 1750 r.p.m.
3'-0
Wire mesh safety screen
18'-6"
Supply tank for sprays 16 foot head
City water supply, float valve control.
STATION No. 1
(Adopted for both 25 and 56 M.P.H. wind)
Horizontal section
Spray shield
Test wall asphalt sealed
Spray pipe slot cover
60° Propellor - hub designed to accomodate 40° propeller for 25 m.p.h. wind also.
Thrust bearing
Propellor ring
Splitter unit
Access panel
Motor stand anchored in concrete
ELEVATION AND SECTION A-A OF STATION No. 1
AIR ENTRANCE END
Splitters
Counter balanced spray pipe holder
1" Shaft
2x12x36" Wood upright
¾" Aluminum spray pipes for various rain intensities - holes drilled with No. 65 drill
1½x1½x¼ L Braces
¾ HP motor 1750 r.p.m.
Plywood spray pipe slot cover
Curved slot for spray pipes
2" Pulley
1½" Water manifold
Wood base 2 pcs. - 2"x12"x36" anchored to concrete base
Wind and rain
Concrete base
Connecting rod
12" V Belt pulley 1" shaft
Flexible rubber hose
OSCILLATING SPRAY

FIG. 4—RAIN RESISTANCE TESTING APPARATUS

Application of Electrodes

Copper strip electrodes to serve later in detecting moisture penetration were arranged on the back face of all walls and also in the core spaces of the concrete masonry walls.

Storage and Age of Walls Before Testing

All walls were stored under cover during the curing period with artificial heat being employed when necessary in order to dry the masonry to the desired value. Brick walls were at least 80 days old at time of test. Those built of high lime and straight lime mortars were allowed to cure a minimum of 100 days before being tested. The concrete masonry walls were allowed to age at least 4 weeks before testing.

TESTING APPARATUS

Fig. 4 shows the principal details of the testing apparatus developed for the purpose of subjecting the specimens to selected and controlled combinations of wind and rain intensities. It differs only in certain details from the apparatus described in the progress report. One duct was enlarged to accommodate a 60-in. propeller producing a 56 m.p.h. air stream as well as a 40-in. propeller producing a 25 m.p.h. wind. The oscillating spray which was incorporated in 1936 proved to be a decided improvement over the fixed sprays used in 1935.

The apparatus was carefully calibrated and throughout the investigation the desired wind velocities and rain intensities were maintained within close limits. Several check calibrations were made during each season's operation.

The wind and rain duct was placed at a 45 degree angle with the wall face because of the practical necessity of diverting the rebounded rain away from the wall and apparatus. Then too, it appeared that natural rainstorms usually impinge angularly with the wall face due to the natural shifting of the wind and directional interference caused by adjacent structures, trees and topographical irregularities.

Wind and Rain Intensities

The selection of a 25 m.p.h. wind velocity as standard in these tests was based on a rather comprehensive survey of climatic data pertaining to the following cities: Chicago, Cleveland, St. Louis, New York, Washington, Jacksonville, Florida, Galveston, Texas, New Orleans and Miami. The records studied covered all rains of 1 in. or more in 24 hours which occurred during the 12 to 20-year period ended in 1935. These data showed that of the total of 1,759 such rains all but 36 were accompanied by winds whose velocity averaged less than 25 m.p.h.

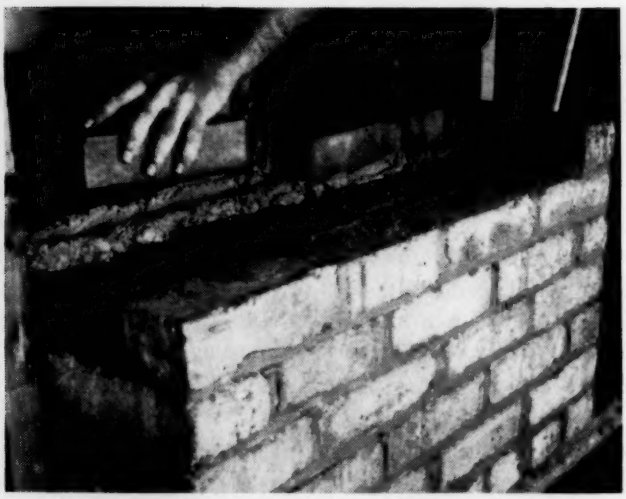


FIG. 5—AVERAGE WORKMANSHIP; 4 IN. FACE BRICK WITH 8 IN. COMMON BRICK BACK-UP

FIG. 6—AVERAGE WORKMANSHIP; 4 IN. FACE BRICK WITH 8 IN. CINDER CONCRETE BLOCK BACK-UP

Concerning the duration of rainstorms, the records showed that continuous rainfall seldom exceeds 24 hours but that intermittent rainfall (with less than 8 hours' cessation) sometimes lasts for two to four days. Invariably, however the longer rains are of much lighter intensity. Intensities exceeding 1 in. per hour seldom last for more than a few hours. Series G test results indicated that the 24 hour

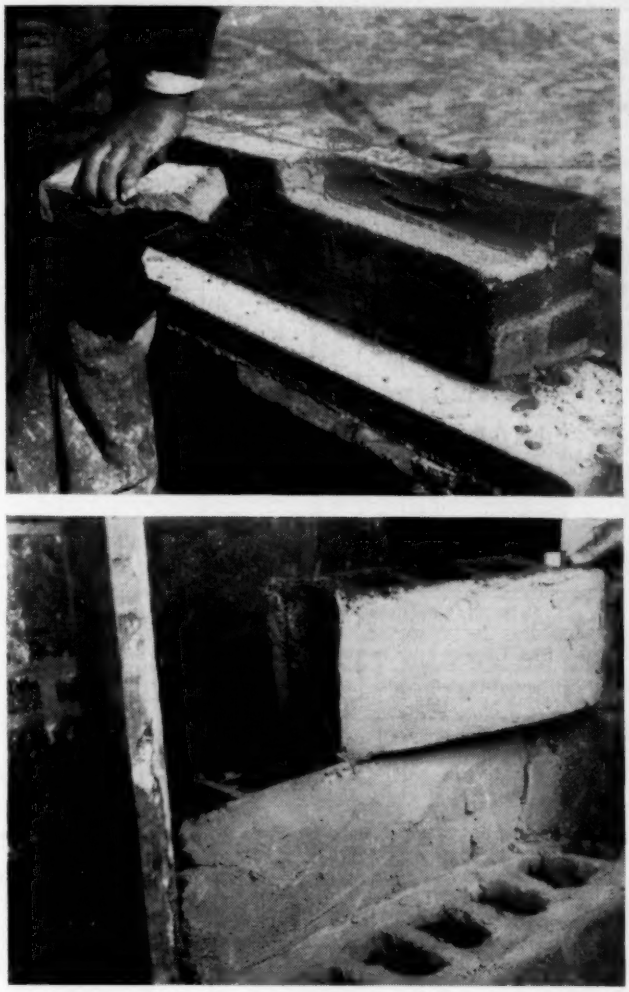


FIG. 7—EXCELLENT WORKMANSHIP; 4 IN. FACE BRICK WITH 4 IN. COMMON BRICK BACK-UP

FIG. 8—EXCELLENT WORKMANSHIP; 4 IN. FACE BRICK WITH 8 IN. CINDER CONCRETE BLOCK BACK-UP. BOTH THE BRICK AND CONCRETE BACK-UP WERE PARGED WITH MORTAR AND ADDITIONAL MORTAR WAS "SLUSHED" IN LONGITUDINAL JOINT TO INSURE SOLID FILLING

TABLE 4—PHYSICAL PROPERTIES OF MORTARS USED IN BRICK WALLS; SERIES F, F1, F2 AND F3

Wall Code No.	Type of Mortar	Mix by Volumetric (1) Proportions C—L—S	Compressive Strength; (2) p.s.i. 28-Da.	6-Mo.	Absorption 24 Hr. Immersion % of Dry Wgt.	% of Volume	Water Retention % (3)
Series F (Lime Used Was a Dry Hydrate Added Dry)							
F1, 2, 13, 14	High Cement	1–0.25–3	2,570	—	10.3	20.0	45
F3	" " (4)	1–0.25–3	1,580	—	1.9	3.6	49
F4, 5, 15, 16	Cement-Lime	1–1 –6	840	—	11.1	21.3	36
F6	" " (4)	1–2 –6	640	—	1.7	3.1	51
F7, 8, 17, 18	High Lime	1–2 –9	510	—	11.8	22.1	42
F9	" " (4)	1–2 –9	430	—	1.2	2.1	50
F10, 11	Lime	0–1 –3	110	—	10.6	19.8	42
F12	" (4)	0–1 –3	70	—	1.1	1.9	66
Series F1 & F3 (Lime Used Was a Presoaked Hydrate)							
F1-1, 2, 13, 14	High Cement	1–0.25–2.5	3,060	2,910	10.6	20.8	59
F1-3	" " (5)	1–0.25–2.5	3,020	3,070	8.3	16.2	68
F3 & F1-4, 5, 15, 16	Cement-Lime	1–1 –5	1,000	1,170	13.2	24.3	72
F1-6	" " (5)	1–1 –5	920	860	11.4	20.8	76
F1-7, 8, 17, 18	High Lime	1–2 –7.5	450	640	13.9	25.4	76
F1-9	" " (5)	1–2 –7.5	390	520	12.9	23.1	79
F1-10, 11	Lime	0–1 –2.5	100	220	12.2	20.8	84
F1-12	" (5)	0–1 –2.5	140	240	11.0	18.5	86
Series F2 (Lime Used Was a Quicklime Putty)							
F2-1, 2, 13, 14	High Cement	1–0.25–2.5	2,570	2,960	11.8	23.1	68
F2-3	" " (5)	1–0.25–2.5	2,680	3,010	9.5	18.5	68
F2-4, 5, 15, 16	Cement-Lime	1–1 –5	790	950	14.5	26.6	78
F2-6	" " (5)	1–1 –5	1,080	1,380	10.1	18.5	79
F2-7, 8, 17, 18	High Lime	1–2 –7.5	410	630	15.0	26.6	86
F2-9	" " (5)	1–2– 7.5	310	420	14.0	24.3	87
F2-10, 11	Lime	0–1 –2.5	70	160	12.1	20.8	92
F2-12	" (5)	1–1 –2.5	80	160	12.5	20.8	95

NOTES: (1) Based on cement at 94 lb./c.f. and dry rodded sand at 106 lb./c.f. in all series; lime at 40 lb./c.f. in Series F, F1, F3; lime putty at 86 lb./c.f. in Series F2.
(2) Values are averages of 3 2-inch cubes tested air dry.
(3) Flow after 60 seconds suction expressed as percentage of flow immediately after mixing. Determinations made in accordance with Federal Specification: Cement; Masonry (SS-C-181 A).
(4) 2 lb. calcium stearate added per 2½ c.f. dry mortar material.
(5) 0.5 lb. calcium stearate added per 2 c.f. dry mortar material.

test with 2½ in. rain intensity is equivalent to a much longer period with a ½-in. rain.

The test exposure first employed in series A consisted of 24 hours with a 25 m.p.h. wind and 2½-in. rain intensity. The unpainted concrete masonry walls developed appreciable leakage well within the 24-hour period. However, in testing the painted walls in this series very little penetration was obtained. Therefore, the cycle was changed to 12 hours with 2½-in. rain intensity and 25 m.p.h. wind immediately followed by another 12 hours with 12-in. rain intensity and 25 m.p.h. wind. This cycle was employed in testing all painted concrete masonry

TABLE 5—PAINT APPLICATION DETAILS SERIES D CONCRETE MASONRY WALLS

Walls built of 8 by 8 by 16 inch 3-oval core, 36.5% core space, cinder concrete block, laid in 1-1-6 cement-lime mortar with face shell mortar bedding and concave tooled joints.

Walls D-1 to D-8 painted with job-mixed compositions, walls D-9 to D-15 with proprietary paints.

Cement used in job-mixed compositions was commercial white portland cement; lime-commercial hydrated lime; sand—fine, white silica sand; waterproofing—powdered calcium stearate.

Wall surface wet down before painting. All walls brush painted with stiff bristle brush except D-14 and 15 which were spray painted. Each coat damp cured 2 days. Walls at least 14 days old before painting.

Wall Code No.	Paint Designation	Dry Proportions (1) C—L—S	Gauging Water, Per 10 Lb. Dry Material: Qts.		Coverage—Dry Material Used Per 100 Sq. Ft.; Lb.	Application Time Per 100 Sq. Ft. Hr.—Min.	Relative Paint Consistency
			First Coat (2)	Second Coat (2)			
D-1	A	1—1—2	1.85	2.05	32.5	1—15	Thick Cream
D-2	B(W)	1—1—2	2.05	2.05	28.5	1—05	" "
D-3	C(W)	1—0—1	1.7	1.7	30.0	1—10	" "
D-4	D	2—1—1	2.2	2.2	27.0	1—05	" "
D-5	E(W)	3—0—1	2.0	2.0	28.3	1—05	" "
D-6	F	3—0—1	2.0	2.0	26.5	1—05	" "
D-7	G(W)	1—0—0	2.9	2.9	21.2	0—55	" "
D-8	H	1—0—0	3.0	3.0	21.3	1—05	" "
D-9	Me	(82%)	5	4	14.5	1—24	Thin
D-10	Bo	(46%)	6	6	9.5	1—10	Very Thin
D-11	Ta	(55%)	4	4	15.8	1—58	Thin
D-12	Ma	(44%)	3	3	18.0	1—20	Slightly Thin
D-13	Ze	(49%)	4	4	14.0	1—45	Thin
D-14	Co	(27%)	0.83	–	95.0	0—33	Thick
D-15	Co	(27%)	0.88	–	102.0	0—37	Thick

NOTES: (1) Volumetric proportions based on cement at 94 lb./c.f., lime at 40 lb./c.f., sand at 115 lb./c.f., given for job-mixed paints. Approximate portland cement content of the proprietary paints is given in percentages by weight. These paints also contained various amounts of hydrated lime, sodium chloride, silica sand, barium sulphate and other ingredients. Paint Co contained a large amount of marble sand.

(2) Proprietary paints were gauged according to manufacturers' instructions.

(W) Waterproofed—2% by weight of cement plus lime.

walls and for series F, F1, F2, H and J. Some of the tests in series F3 were of longer duration and in Series G different combinations of rain and wind intensity were employed.

The rain intensities employed in the tests are expressed in inches of water per hour striking against the vertical face of the specimen; that is, they are equivalent to the depth of water which, if allowed to remain, would accumulate in one hour on the wall face. They are not necessarily equivalent to natural rainfalls of the same intensity as these are customarily measured and expressed in inches of water falling on a horizontal surface. The relation which the intensity on a vertical surface bears to that on a horizontal surface depends upon the vertical angle at which the rain falls as determined by the velocity of the wind and other factors.

TEST PROCEDURE

The test wall was set up in position as indicated in Fig. 4 with the spray shields clamped in place and the joint between the wall and shield tightly caulked with hot asphalt. If weight readings were to

TABLE 6—APPLICATION DETAILS OF SURFACE WATERPROOFING COMPOUNDS USED ON SERIES H WALLS

Walls built of 8 by 8 by 16 inch 3-oval core, 36.5% core space, concrete block, laid in 1-1-6 cement-lime mortar with face shell mortar bedding and concave tooled joints.
Block at least 28 days old when walls were erected. Walls at least 14 days old before applying surface treatments. A 24-hour drying period was allowed between coats.
Compounds were applied with a stiff hair brush.

Wall Code No.	Type of Concrete Block	Compound Designation (1)	Amount of Compound Applied Per 100 Sq. Ft.; Gallons				Coverage Per Gal. Sq. Ft.
			1st Coat	2nd Coat	3rd Coat	Total	
H-1	Cinder	Mn	2.08	1.57	——	3.65	27.4
H-2	Limestone	Mn	1.72	1.38	——	3.10	32.3
H-3	Cinder	C.D.	0.71	2.7	1.87	5.28	19.0
H-4	Limestone	C.D.	1.09	2.04	1.98	5.11	19.6
H-5	Cinder	M#1	1.42	1.78	2.33	5.53	18.1
H-6	Limestone	M#1	1.78	1.95	1.58	5.33	18.8
H-7	Cinder	M#2	1.09	0.96	0.72	2.77	36.1
H-8	Limestone	M#2	0.98	0.93	0.97	2.78	36.0

NOTE: (1) Mn was a heavy bodied tung oil paint. The other compounds were thin bodied liquids of the penetrative type usually consisting of oil, wax, stearate or resins in a solvent.

be taken the wall and shield were supported on a platform scale; otherwise, concrete block supports were used.

Initial observations and readings were made following which the wind and rain were turned on. Thereafter these data were taken as frequently as conditions dictated depending on the porosity of the wall and the anticipated rate of penetration.

Moisture penetration was observed by three different methods. An instrumental method, involving resistance readings resulting from passing a constant voltage current through the copper strip electrodes, was employed throughout. This method was described in the progress report. Probably the most significant information was obtained from the frequent visual examinations. Comprehensive notes were recorded pertaining to the source, location, amount and kind of moisture penetration. In describing the latter a distinction was made between moisture absorption (penetration absorbed by and retained in the masonry) and fluid leakage (water actively exuding from the wall and draining down the pallet).

In all of the brick wall series and some of the concrete masonry series the amount and rate of penetration was determined by weight as supplemental to the other observations. This was accomplished simply by supporting the test wall with shield on a suitable platform scale, weighing at time intervals and making the proper deductions for the shield and dry wall weight to obtain the weight gain due to moisture absorption. Fluid leakage was collected in a receptacle and weighed and recorded separately.

SUMMARY OF TEST RESULTS

The practical value of the findings depends upon their applicability to commercial masonry practice. While the test conditions may have differed in some respects from natural conditions, yet fundamentally they were much the same. Therefore, it would seem that constructions which performed well under test will provide relatively greater weathertightness in buildings than those types which leaked badly in the tests. It also follows that the various factors studied will exert substantially the same influence in the case of actual buildings and masonry practice as they did with respect to the test walls.

Performance Rating of Walls

The main wall groups exhibited sufficiently different average performance characteristics to permit their being broadly classified as to rain resistance.

In the following references to mortars, class A, class B and class C mortars are considered to be roughly comparable to the mortars used respectively in series F2, F1 and F.

Excellent Performance—Groups which on the average developed no fluid leaks or back face dampness in 24 hours of testing were:

Painted concrete masonry walls, the paint coatings of which were thoroughly applied with stiff bristle brush to form a continuous film free from voids or significant crazing.

Concrete masonry with three coats portland cement stucco.

Cast-in-place concrete walls.

Eight inch solid face brick walls built with excellent workmanship and class A mortar.

Twelve-inch face brick walls with either solid brick or concrete masonry back-up built with excellent workmanship.

Good Performance—Groups which on the average developed no fluid leakage and a dampening of less than 25 per cent of the back face in 24 hours of testing were:

Eight-inch solid face brick walls built with average workmanship and class A mortar or with excellent workmanship and class B mortar.

Painted concrete masonry walls, the paint coatings of which had developed superficial crazing due to normal shrinkage effects.

Fair Performance—Groups which on the average developed not more than 15 lb. of fluid leakage nor a dampening of more than 50 per cent of the back face in 24 hours of testing were:

Eight-inch solid face brick walls built with average workmanship and class B mortar or with excellent workmanship and class C mortar.

Eight-inch walls of 2-in. face brick and 6-in. concrete masonry back-up built with through header courses (no header block).

Spray painted concrete masonry walls, the coatings of which contained minute pinholes.

Eight-inch solid select common brick walls built with excellent workmanship and class B or better mortar.

Twelve-inch walls of 4-in. face brick plus 8-in. common brick or concrete masonry back-up built with average workmanship and class B mortar.

Poor Performance—Groups which on the average developed from 15 to 50 lb. fluid leakage and/or a dampening of 50 to 75 per cent of the back face in the 24-hour test were:

Painted cinder block walls, the paint coatings of which were badly crazed due to excessive expansion of the cinder concrete.

Eight-inch select common brick walls built with excellent workmanship and class C mortar.

Very Poor Performance—The performance of the following groups was characterized by rapid penetration resulting in fluid leakage of over 50 lb. and usually a general dampening of the back face:

Concrete masonry walls unpainted or treated with thin-bodied liquid waterproofings of the penetrative type.

Eight-inch solid common brick walls built with average workmanship.

Eight-inch walls of 4-in. face brick and 4-in. cinder tile back-up built with average workmanship.

Eight-inch walls of 2-in. face brick and 6-in. concrete masonry back-up built with header block at header courses and with no provisions to retard the downward drainage of moisture in the core spaces.

CONCLUSIONS

1. The unpainted concrete masonry walls leaked rapidly regardless of type and grading of aggregate, concrete mix or surface texture employed in the concrete units. Much of the leakage occurred at the mortar joints.

2. As compared to the plain face concrete masonry units, the oscillated face units were slightly less permeable, the water eroded face units slightly more permeable.

3. All concrete masonry walls given two coats brush applied portland cement paint were practically impervious as long as the paint remained in good condition. In general, the paint proved slightly more effective when used on block having relatively smooth, dense surfaces.

4. One coating of paint greatly improved the performance of concrete masonry but was less resistant than two coats.

5. Sprayed coatings of cement paint developed small pinholes through which moisture penetrated and they proved less effective than brush applied coatings.

6. The 4-in. painted concrete masonry performed satisfactorily but somewhat less well than the 8-in. painted walls.

7. Equally good results were obtained throughout a wide range of job mixed and commercial brands of portland cement paint. Mixtures containing a large proportion of lime were however, inclined to become chalky and their durability is questionable.

8. The stuccoed concrete masonry wall showed no penetration through either the exposed face or back face after 219 hours of test exposure.

9. Painting over the joints slightly reduced the leakage through plain concrete block masonry.

10. In tests of Haydite block walls leakage was not affected by the use of face shell as compared to full mortar bedding.

11. The plain concrete masonry walls of series J built of high pressure steam cured block leaked appreciably less than the other plain walls. Aside from the difference in curing, series J block were extremely dense and smooth on the surface.

12. The application of three coats of thin-bodied, penetrative type waterproofing compounds was totally ineffective in sealing the surface of concrete masonry walls, due to their inability to form a continuous, moisture resistant film on the surface or to fill and close the relatively large voids and capillaries beneath the surface.

13. A normal amount of crazing due to shrinkage and weathering effects over a 32 months' period did not seriously impair the imperviousness of portland cement paints. However, cracking caused by abnormal volume change in the concrete of some walls resulted in rather rapid penetration in the retests.

14. Paints applied on limestone or sand and gravel block developed little crazing in 32 months of weather exposure. Crazing was more evident on the painted Haydite and cinder block walls of series A, but it did not greatly impair wall performance. The paints applied on series D cinder block walls crazed very extensively and caused these walls to retest poorly following a 12 months' weather exposure period. Crazing appeared tó be induced by excessive expansion of the masonry which was ascribed to unsound qualities of the particular aggregate used. Further studies involving the weather exposure

of painted walls made of cinder block obtained from different manufacturers indicated that the crazing experienced in series D was much worse than will occur with average commercial cinder block. These latter results indicated that properly made cinder concrete masonry can be painted with good results but that manufacturers should scrupulously avoid using cinders of unknown or questionable quality.

15. The weathering of series D walls caused all of the various paints to craze and no advantage of one over the other or of job mixed as compared to prepared paints could be determined. Further studies have shown that suitable job mixed compositions compare favorably with the better prepared paints for use on sand and gravel or limestone block, but some prepared paints have proved slightly superior for use on cinder, Haydite or slag aggregate block. Most of the prepared paints studied retain their opacity better when wet than do job mixed compositions, presumably due to the inclusion of a small amount of some opacifying agent, to finer grinding, or both.

16. The performance of the brick walls ranged from very satisfactory to very poor. Of 36 8-in. face brick walls tested, 29 were penetrated to the back face. Considering the 54 brick walls in series F, F1 and F2, penetration ranged from 11.7 to 756 lb. of water.

17. The quality of workmanship was the most important factor influencing the performance of the brick walls. Excellent workmanship and high rain resistance requires the complete and careful filling of all joints so that the masonry is a solid mass free from cavities and interstices through which moisture can seep.

18. Wall performance was influenced by the kind of brick used. Face brick and select common brick being relatively true in shape and size and free from warpage, chipped corners and edges facilitated the complete filling of joints and thorough enclosure of the brick in mortar.

19. The absorptive properties of the brick affected laying somewhat but with suction controlled by dampening when necessary presented no particular difficulty.

20. Mortar composition with respect to the cement-lime ratio had no consistent effect on wall performance. No improvement was obtained either as to workability from the mason's viewpoint or as to wall performance by increasing the proportion of the lime within the range of mortar mixes studied.

21. In general, the addition of calcium stearate waterproofing to the mortars increased their plasticity and improved wall performance.

This effect was less with respect to the mortars employing more plastic lime, as in series F2.

22. The quality of the lime used in the mortar appreciably influenced mortar workability and wall performance. The dry hydrated lime used in series F was the poorest, the quicklime putty used in series F2 was the best, and presoaked hydrated lime used in series F1 was of intermediate quality. The water retentivity of the high cement content mortars of series F1 and F2 was higher than that of the straight lime mortar of series F. In this latter series, water retentivity was not improved by increasing the proportion of dry hydrated lime to cement.

23. The 12-in. face brick walls were substantially less permeable than similarly built 8-in. walls when excellent workmanship was used but were little, if any, superior when built with average workmanship.

24. The use of header block of the open core space type reduces the effectiveness of concrete masonry back-up in retarding the gravitation of moisture and superior wall performance will be obtained by backing up the face brick headers with a brick stretcher course or adopting other means of closing the core spaces of the block at intervals.

25. Face brick walls having concrete masonry back-up laid with open interior vertical joints leaked much more rapidly than when these joints were filled by "buttering" the edges of the block. The use of open interior joints is a practice which should be strenuously discouraged.

26. Twelve-inch face brick walls having concrete masonry back-up performed as well as those built with common brick back-up.

27. The use of concrete masonry as compared to common brick for back-up reduced mason time and mortar requirements about 25 per cent in building the 12-in. walls.

28. The construction consisting of 4-in. face brick backed up with 4-in. cinder tile laid with average workmanship, leaked very rapidly. It would appear that in practice this type wall should be built with excellent workmanship.

29. The walls built of 2-in. face brick performed quite poorly when used with 6-in. concrete masonry back-up having so-called "header block" for bonding the 4-in. header brick. In this construction moisture which penetrated the facing wythe collected on the 4-in. brick header and there was diverted into the core space where it drained freely to the pallet, causing pools of water and excessive base

joint leakage. There also was considerable penetration to the back face at the joints below the header course due to the moisture which gravitated downward and inward from the header joint.

30. The 2-in. brick faced walls performed more satisfactorily when through header courses were employed. The through headers reduced the amount of collected penetration at any one place and tended apparently to diffuse the penetrated moisture more evenly throughout the back-up.

31. Walls that exhibited relatively impervious qualities in the standard test were also highly resistant in the test employing a 56 m.p.h. wind. In general, varying the rain and wind intensities had little effect on the performance of walls having low surface permeability. Walls having relatively pervious surfaces, such as unpainted concrete masonry or the brick walls (by virtue of the joints), were penetrated more rapidly with an increase in either rain or wind intensity.

32. The 4-in. thick reinforced concrete walls performed very satisfactorily under all wind and rain intensities used. Moisture absorbed ranged from 1.5 to 8.0 lb. in 24 hours. All walls remained dry on the back face throughout each test.

33. Hermetically sealing the top and sides of the test walls appeared to increase the rate of rain penetration somewhat but not to a degree requiring any adjustment of the test results.

Discussion, to close in April, 1940 JOURNAL, *should reach A. C. I. Secretary in triplicate by Feb. 1, 1940.*

Vol. 36 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE
(Supplement)

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *SEPTEMBER 1940*

Discussion of a paper by R. E. Copeland and C. C. Carlson:

Tests of the Resistance to Rain Penetration of Walls Built of Masonry and Concrete*

BY D. E. PARSONS, BENJAMIN WILK, E. W. SCRIPTURE, JR. AND AUTHORS

CONVENTION DISCUSSION

D. E. Parsons†—Many have looked forward to this opportunity of becoming familiar with the recent progress of the investigation first described in 1936. The expectation that the final results of this important and comprehensive study would be of great practical value has been fully justified. It is now apparent that the investigation was carefully planned and skillfully executed. The exposures of the walls closely simulated the conditions which exist during a heavy rain accompanied by a strong wind, and the performances of the walls, therefore, should be indicative of the relative watertightness of building walls under natural exposures.

In an investigation which has been under way at the National Bureau of Standards for the last four years more than 400 wall specimens have been tested. The general method of investigation was similar to that of the authors, but the method of testing differed. The walls were approximately 50 in. high and 40 in. wide. They were built in the laboratory by skilled masons. The variable factors for the brick masonry included types of brick, composition of mortar and method of forming joints. Walls of hollow units were faced with brick masonry, portland cement stucco or cement paint. The walls were exposed to conditions simulating the effects of wind-driven rains by subjecting one face to a water spray and a static air pressure. During an exposure a wall was clamped against one face of an air-tight chamber or box,

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†National Bureau of Standards, Washington, D. C.

the wall forming one side. Water was applied at a rate equivalent to 5.5 in. per hour over the exposed area of the wall, while an air pressure of 10 lb. per sq. ft. was maintained within the chamber. This pressure is approximately equivalent to the maximum of a 50 miles per hour wind. The performances of the walls were judged by noting the time for dampness and free water to penetrate, the rate of leakage and the proportion of the area of the unexposed face becoming damp in 24 hours.[1]

Although the two investigations were not identical in scope, the studies were similar in several particulars and for most of these the results are in excellent agreement. The concrete masonry walls without facings, those with facing of poorly built brickwork, and the walls of brick masonry with inferior workmanship showed very poor performances. Excellent performances were shown by walls of concrete masonry faced with portland cement stucco or with cement paint applied by means of stiff-bristled brushes and by walls of brick masonry with excellent workmanship. Variations in the composition of the cement paint within the range used did not have a significant effect. Thin-bodied colorless waterproofings were ineffective in sealing openings. The quality of the workmanship was the most important factor affecting the performance of the brick walls. The performance of the brick walls was not related to the cement-lime ratio of the mortar but depended in part on the water retentivity of the mortar. The authors appear to have obtained somewhat better performance with 8-in. face brick walls built with an average workmanship than was obtained in the bureau's investigation and somewhat less difference between the 12-in. and the 8-in. brick walls.

One of the series of tests at the Bureau was for the purpose of determining the effects of the absorptive properties of the bricks, the water retentivity of the mortar, and the method of forming joints on the water permeability of brick walls. The bricks were of a wide range in absorptive properties, the more absorptive ones being laid dry and also with varying amounts of moisture. Different methods of forming the joints were tried. The cement-lime mortars covered a wide range in water retentivity as determined by the flow after suction test. A consistent relation was found between the suction rate (rate of absorption)[2] of the bricks at time of laying and the water permeability of the walls. The walls built with well filled joints and bricks having suction rates greater than 50 grams per brick per minute were rated

[1] A detailed description of the methods of testing is given in the report, "Water Permeability of Masonry Walls," National Bureau of Standards BMS 7 (1938).

[2] The suction rate of a brick is the amount of water absorbed by the brick when immersed flatwise to a depth of 1/8 in. for 1 minute. It may be made as small as desired by wetting the brick before use.

from fair to very poor; those built with bricks having suction rates between 20 and 50 grams per brick usually were rated good or fair. All walls built with brick having suction rates of 20 grams per brick or less showed a satisfactory performance, most of them being rated excellent, a few good and only one fair. Thus the results of the tests indicated that, regardless of the kind of brick or mortar, the permeability of the walls decreased with a decrease in the suction rate of the brick. This suggests that the performance of the authors walls of common brick and excellent workmanship might have been better if the bricks had been wetter when laid.

To summarize the conditions governing the water permeability of brick masonry facings, the methods of laying brick which resulted in highly resistant walls were those providing either solidly-filled vertical joints or a barrier consisting of a continuous coating of mortar within the interior of the walls. The likelihood of obtaining leakage resistant joints was greatest when the bricks were as wet as they could be without excessive floating and the joints were filled with a water retentive mortar.

A few of the walls, after being tested for water permeability, were subjected to 10 cycles of wetting and drying; others to 30 cycles of heating to a temperature of 125°F., followed by cooling to 20°F. Upon retesting to determine the change in permeability, it was found that neither of these exposures had an important effect on water permeability. The rest of the walls were stored outdoors for periods up to three years and then retested. In general the exposure did not result in marked changes of the permeability of the walls.

(Note: Following the presentation of the foregoing written discussion Mr. Parsons added the following remarks).

I was particularly interested in Mr. Copeland's description of the workmanship. I would like to ask whether or not he had any difficulty in filling the vertical joints between the facing brick and the hollow unit? Our experience has been that when a mason uses the technique that he is likely to use in actual construction, he does not obtain a completely filled joint by attempting to slush in mortar from above, and that the likelihood of satisfactory performance is greater if he concentrates his attention upon the quality and completeness of a plaster coat of mortar applied either to the back of the facing or to the face of the back.

It seems to me that the problem of obtaining a water-tight joint between mortar and a masonry unit is similar to that of obtaining a water-tight joint in concrete, a construction joint. As all of you know, to obtain a good bond between new concrete and old concrete, you

find it necessary to wet the old concrete and then to apply a brush coating of very wet grout or to place a relatively thicker coating of mortar which is worked into place, and then place the concrete. We have learned by experience that we cannot get a water-tight joint simply by dumping in concrete on to a dry hardened surface. Similarly with masonry units, and particularly with brick masonry, we find it impossible to get a water-tight joint if the suction rate of the brick is too high or if the mortar is too dry and stiff at the time the contact is made. It is impossible to form a complete contact between the mortar and the unit unless both the mortar and the unit are in proper condition. That, I think, is one reason why, with the type of workmanship which Mr. Copeland has designated as average workmanship, we find that the vertical joints leak very rapidly. In applying the mortar to the edge of a facing brick, the only mortar that the mason uses is that which he scrapes from the face of the wall and which has lost a part of its water to the adjacent brick. That mortar at the time the mason applies it to the joint, is too dry to form an intimate and water-tight contact with the brick. Another illustration of the same phenomenon is an experiment which we tried in which we constructed the wall with workmanship similar to that described as average, and then, in an attempt to seal the joints which we knew to be defective, we had the mason tool those joints very carefully after the mortar had stiffened sufficiently to be in what we thought and he thought was the right condition. A visual examination of those joints after construction indicated that a complete joint had been formed on the surface, and yet the results of the permeability test showed that, while the leakage had been reduced, there was still considerable leakage. After tearing down the wall and examining the joints carefully with a magnifying glass, we found there had not been a complete and intimate contact with the mortar at the face of the wall and that the mortar presumably was too dry and stiff to make an intimate contact.

*Benjamin Wilk**—The paper shows that portland cement paint applied to concrete masonry surfaces is an excellent water repellent. This is of considerable interest to concrete products manufacturers and builders. Experience shows that one of the big problems in the construction of concrete masonry houses is what to do about the surface. The interest that has been shown in painting houses, even those built of brick, shows a trend that is beneficial in considering concrete masonry.

The possibility of rain penetration through a masonry wall is more likely to occur in the mortar joints than through the block itself. We

*General Manager, Standard Building Products Co., Detroit.

have been suggesting for several years that concrete masonry walls exposed to the weather be painted. We have found that paints having a portland cement base have been effective not only in repelling water but also in giving an attractive surface. The interest in painting concrete masonry walls has created a number of paints with various bases. Tests made at the Bureau of Standards, and reported recently at a meeting of the National Concrete Masonry Association by Mrs. Clara Sentell, show that effective paints can be produced but that improvement is still possible.

One of the big troubles is that the average painter knows very little about the application of paint to masonry surfaces. With the portland cement base paint the walls should be dampened before the application of the paint and after the paint has been applied the walls should be kept damp for several days, especially in warm weather. In applying cement base paints a scrub brush, rather than a paint brush, has been recommended for applying the first coat so that the paint will be scrubbed into the surface filling up even the pin holes. The second or finish coat can be applied with a short paint brush. Thorough coverage is important. If these details are not followed poor results are to be expected. Some products manufacturers prefer to use ordinary laborers rather than painters in applying paints to masonry walls because they can be told what to do and are more likely to follow instructions. Where oil base paints are used there is danger of peeling if moisture gets behind the paint.

Products manufacturers feel that within a comparatively short time commercial paints will be available for application on concrete masonry walls that will not only be attractive and water repellent but will also eliminate the streaking which is more or less noticeable on some painted concrete masonry walls.

*E. W. Scripture, Jr.**—This paper throws light on a number of moot questions in masonry construction, and raises others. These deal particularly with portland cement paints and with the materials of construction.

The authors have shown that all unpainted concrete masonry and cinder block walls leaked badly, also that proper treatment with portland cement paint reduced leakage in all cases and in many cases made the walls substantially weather-proof. Some points in connection with cement base paints may be mentioned.

A cement base paint consists primarily of portland cement, usually white. It is necessary that the cement used be finely ground and

*Master Builders Research Laboratories, Cleveland.

develop high strength fairly rapidly and for these reasons as well as the matter of color, a fine ground white cement is used. It is also desirable to incorporate in the paint a pigment, usually white, to give a better color and higher opacity when wet, and the authors note this quality in prepared cement paints compared with job mixed paints. The tendency of cement paints to craze is also noted by the authors and to minimize this tendency it is desirable to incorporate in the paint small amounts of materials which will (1) retain moisture in the paint after it is applied and (2) prevent too rapid setting. It is also desirable that the cement paint form a readily workable paste with water and for this purpose, as well as lower cost, some cement paints contain considerable proportions of lime. This is undesirable, as the authors have shown, for the simple reason that the higher the proportion of lime in the paint the more like white-wash it becomes, that is, the less its strength and durability. The inclusion of a small amount of stearate in the paint helps the workability, improves the opacity when wet, and reduces the rate of penetration of moisture, but the amount of stearate should not be excessive. The requirements of a cement paint are: 1. Good color, 2. Finely ground cement, 3. Pigment, 4. Retention of water and slow setting, 5. Minimum of lime or none at all, 6. A small amount of stearate.

It is believed that these ingredients can best be combined in a prepared paint and that the properties of a good prepared paint will be definitely superior to a job mixed paint. Where a colored paint is desired it is practically impossible to secure uniform distribution of the color on the job; ball mill or other type of grinding is required. One reason the authors failed to find more marked differences between the prepared and job mixed paints may be that in this experimental investigation the job mixed paints were probably much more carefully prepared and applied than would be the case in practice. Another reason may be that some of the prepared paints which they investigated were of high quality whereas others were not.

Some other points in connection with cement paints brought out in this paper are:

1. All concrete masonry should be painted with portland cement paint or stuccoed.

2. Spraying is not as satisfactory as brushing. This is to be expected since a higher w/c is required for spray painting. The authors also note pitting of the paint film when sprayed.

3. Painting of the joints slightly reduced leakage. We have found this very effective where the application was made carefully, scrubbing

the paint into openings such as cracks, particularly where the wall was of rather poor quality.

3. Transparent waterproofings were ineffective. This is entirely in accord with a fairly extensive investigation which we have made of this subject in which we found these materials effective to a very limited degree only in the first instance and entirely lacking in durability.

5. The authors' conclusion 13 that crazing did not seriously impair the imperviousness of portland cement paints seems inconsistent with their prior statements of results, i.e., Excellent performance, Painted concrete masonry walls and Good performance, Painted concrete masonry walls which had developed superficial crazing. This seems like a significant difference and would probably become more important with time.

6. We have also found cinder block difficult to paint satisfactorily but have found that a priming treatment with metallic waterproofing or a cement-sand slush coat or both permits satisfactory application of cement paint.

It seems somewhat peculiar that when the painting of masonry walls is investigated, the investigators confine themselves either to cement-base paints or to oil paints. Last year a paper was presented on various compositions of oil paints for masonry without any comparison with a cement base paint. In this paper the authors state that certain assemblies (Series I) were painted with oil paint but I do not find any mention of the relative performance of these two types of paint. There has also been no investigation of the metallic type of waterproofing which has been found very effective in a large number of instances in the field, simply as a waterproofing coating per se, as a priming treatment before application of cement paint as previously mentioned in connection with cinder block, or as a joint treatment.

One point which has been brought out is that of quality of workmanship. It is shown here, as a number of other investigators have shown, that with given materials and type of construction, excellent workmanship will produce much better results than average or poor workmanship. One might also be tempted to go further and say that weathertight walls can be laid up with almost any reasonable combination of materials provided that sufficient care is taken in the laying up. It is not apparent, however, that this has anything to do with what may be expected in the field. Since, as the authors have shown, excellent workmanship compared with average workmanship increases the labor cost 30 per cent and the material cost 20 per cent it is hardly

to be expected that excellent workmanship will be secured on the job. From a practical point of view it would seem necessary to base any conclusions on the results secured with average workmanship.

The authors state that all unpainted concrete masonry units leaked rapidly and that much leakage occurred at the mortar joints. Without detracting from the conclusion that painting such walls is desirable it should be pointed out that this statement is somewhat unfair to concrete masonry as compared with brick when it is considered that these walls were laid up without wetting the units and with a 1:1:6 mortar using dry hydrated lime whereas many of the brick walls which showed comparatively little leakage were laid up with brick which were wetted and using lime putty. It seems quite possible that the mortar used was not suitable for unwetted concrete units.

The statement (conclusion 20) that "No improvement was obtained either as to workability from the mason's point of view or as to wall performance by increasing the proportion of the lime within the range of mortar mixes studied" appears to me perfectly incredible and furthermore inconsistent with the observed wall performance. With respect to the latter, Class A mortars are consistently superior to Class B mortars and Class B to Class C. It is true this is a matter of more plastic limes but that an increase of the proportion of a given lime will not increase the fatness of the mortar is not in accord with our own observations or with those of many others.

Perhaps the authors draw their rather generalized conclusion by giving undue weight to the walls laid up with excellent workmanship whereas these should be disregarded entirely. With sufficient care a tight wall may be laid up with almost any mortar, but it seems to stand to reason that with average workmanship the more plastic the mortar the better will be the result. Another factor involved may be the use of wetted brick and unwetted concrete units. In the first case any of the mortars used might be sufficiently water retentive for the brick and in the second case only one mortar was used. While wetting of the brick is considered good practice by many it does not seem to be common practice at the present time.

Stearates do improve the workability of mortar to a limited extent. The quality of the lime has also been shown to improve water retentivity and other aspects of workability but it is rather surprising that an increase in proportion of lime, even in the form of dry hydrate, does not improve this property. It must have been very poor lime. By inference from Conclusion 22 it would seem that increasing the proportion of the other two limes would increase water retentivity.

Conclusion 28 states that a particular type of construction should be built with excellent workmanship. This would mean to me that this type of construction was unsuitable in practice.

If it is assumed that average or even a little better than average workmanship must be dealth with in the field then weather-tight construction depends on a number of considerations. (1). Type of construction, (2) Quality of units, (3) Quality of mortar, (4) Painting.

As the authors have shown certain types of construction are more desirable than others. Further they have shown that the porosity, surface texture and quality of the units will affect water-tightness. With average workmanship the mortar should be workable and it is reasonable to believe that the more workable it is the better the results will be with a given quality of workmanship. The mortar should have a water retentivity consistent with the porosity of the unit. That these properties should be secured without excessive proportions of lime may be the case but sufficient lime must be used to produce a workable mortar. This points to the desirability of using a highly plastic lime, stearate, and any other means which will secure this result, without using a high w/c. Painting is desirable on porous masonry units for protection quite aside from the question of decoration. It is also desirable on defective joints and in some extreme cases painting should be supplemented with other protective measures.

AUTHORS' CLOSURE

Mr. Parsons' discussion of the extensive investigation at the National Bureau of Standards was most timely. It was gratifying to learn that where comparisons are possible the results of the two investigations are in close agreement.

As Mr. Parsons suggests, it is probably true that the walls built with common brick and excellent workmanship would have performed better if the brick had been more thoroughly wetted before being laid. They were sprinkled with a garden hose, which is practicable to do on the job, and although suction was reduced to where it presented no difficulties it was not controlled to exact optimum limits.

In building walls of face brick with concrete masonry backup using excellent workmanship, the longitudinal joint was filled by plastering the interfaces of both the brick wythe and the backup unit. The latter was plastered before erection and after it was set the joint space was practically filled. Actually very little mortar was slushed into the joint from the top. Although this procedure is more costly than average workmanship it is practicable and produces extremely rain resistant construction.

The use of "scrapings" for the vertical joints is a universal and time honored practice among masons. Its bad effects are particularly pronounced where high absorptive brick and average workmanship are employed and further indicates the need for improvements in masonry practices, at least in regions frequented by hard driving rains.

Mr. Wilk's discussion on painted concrete masonry supplied more evidence that this type wall performs well in service as well as in tests. His remarks on the application of paints agree, in general, with the authors' experience.

Dr. Scripture's remarks concerning portland cement paints contribute valuable information on the subject.

The authors did not conclude, as Dr. Scripture infers, that all concrete masonry should be painted with portland cement paint or stuccoed. The term "concrete masonry" includes many possible types of construction outside the scope of the tests and to which the conclusions would not necessarily apply. For example, walls built of concrete brick or other solid concrete units of substantial thickness would test much better than the unpainted walls of hollow concrete block. Then too, in regions where rains are infrequent and of mild intensity more porous types of walls may be entirely satisfactory.

There is no inconsistency in the statements concerning the effect of superficial crazing of the paint films on rain penetration. This effect was confined to a dampening of less than 25 per cent of the back face with no fluid leakage, which, considering the severity of the test, cannot be construed as a serious impairment of wall performance.

Two walls (H-1 and 2, Table 6), painted with a heavy bodied tung oil paint tested equally as well as the walls having cement paint coats. These results, which were inadvertently not mentioned in the conclusions, indicate that paints of the oil vehicle type, properly applied, should be effective in sealing concrete masonry surfaces. A more extensive study is now being made of the durability of a larger variety of coatings, including oil and synthetic resin types.

Dr. Scripture questions whether excellent workmanship can be economically justified. In the authors' opinion there is no substitute for good workmanship and that the extra cost will be justified where driving rains and leaky walls are a real problem. In other regions, average workmanship should produce satisfactory results. This is a matter that depends upon local climatic conditions and construction practices.

Additional data on the effect of mortar composition, especially with respect to the cement-lime ratio, are shown in the accompanying

TABLE 7—EFFECT OF MORTAR COMPOSITION, TESTS OF 8-IN. BRICK WALLS

(Each value is the average of results on 3 to 6 walls)

Mortar Composition C L S	Face Brick Walls		Common Brick Walls	
	Excellent Workmanship	Average Workmanship	Excellent Workmanship	Average Workmanship
	ENDURANCE PERIODS			
1:0.25:2.5 and 1:0.25:3	17 hr.–15 min.	4 hr.–20 min.	6 hr.–05 min.	0 hr.–59 min.
1:1:5 and 1:1:6	19 hr.–10 min.	9 hr.–40 min.	3 hr.–24 min.	1 hr.–38 min.
1:2:7.5 and 1:2:9	17 hr.–15 min.	9 hr.– 0 min.	11 hr.–13 min.	1 hr.–31 min.
0:1:2.5 and 0:1:3	15 hr.–30 min.	5 hr.–34 min.		
	ABSORPTION AND FLUID LEAKAGE IN 24 HOURS; LB.			
1:0.25:2.5 and 1:0.25:3	20.9	44.4	46.9	418.6
1:1:5 and 1:1:6	26.8	50.7	56.6	451.2
1:2:7.5 and 1:2:9	34.5	43.5	66.8	393.6
0:1:2.5 and 0:1:3	43.8	95.9		

Endurance periods are a measure of the time required for penetration to the back face.

Tables 7 and 8. In Table 7 the results are arranged according to type of brick and workmanship and in Table 8 according to series. These data clearly show that in these tests the cement-lime ratio did not have a consistent effect on wall performance. It should be mentioned that within each workmanship group the different walls were built with the same amount of care and in practically the same time; also that the hydrated lime was a standard commercial brand produced near Chicago, and was purchased under A. S. T. M. specifications. Undoubtedly there are better limes just as there are poorer ones.

TABLE 8—EFFECT OF MORTAR COMPOSITION, TESTS OF 8-IN. BRICK WALLS, BY SERIES*

(Each value is the average of 3 face brick and 2 common brick walls.)

Mortar Composition	Average Endurance Periods; Hr.—Min.			Average Absorption and Fluid Leakage In 24 Hours; Lb.		
C L S	Series F	Series F1	Series F2	Series F	Series F1	Series F2
1:0.25:2.5 and 1:0.25:3	8—42	6—51	11—57	182	97.5	51.8
1:1:5 and 1:1:6	9—18	10—25	12—05	182	131.6	53.3
1:2:7.5 and 1:2:9	4—41	15—16	13—48	179.5	109.3	58.2

*The lime constituent of the mortar was as follows: high calcium hydrated lime added dry in Series F; high calcium hydrated lime presoaked 24 hours in Series F1; high calcium pulverized quick lime putty in Series F2.

Endurance periods are a measure of the time required for penetration to the back face.

Straight lime mortar group omitted as it did not include the same range of wall construction as the other mortar groups.

Table 4 of the paper showed that where the lime constituent was a presoaked hydrate or quicklime putty increases in the lime content increased the water retentivity of the mortar as measured by the flow after suction test.

It is not necessarily true that tighter joints are obtained with the mortar having the higher water retentivity. In fact, in laying low suction face brick high water retention may be detrimental. For general use, a mortar should have a minimum flow after suction and many specifications now include such a requirement. The test data indicate, however, no advantages in increasing the water retention beyond 60 or 65 per cent merely by adding more lime but that results are improved by employing a more plastic form of lime to accomplish the same purpose.

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Forms for Architectural Concrete*

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SYNOPSIS

This paper describes the materials acceptable for forms for architectural concrete and how forms are fabricated and secured for acceptable results.

In a previous paper‡ the requirements for the grade of concrete suitable for architectural concrete were discussed. Equally important to the success of the project is its appearance. Poorly built formwork can prevent a good appearance regardless of the quality of the concrete or the care used in placing. Bulges and irregularities in the contact surfaces, and small leaks in the forms give results that may pass for the ordinary structural work but which would not be satisfactory where the concrete is the architectural medium. Provision for removal of the forms without damage to the concrete must also be considered. This is especially important at moldings and other decorative features.

MATERIALS FOR FORMS

The materials to be used for the contact surfaces of the forms will naturally depend on the effects desired. Common boards, structural plywood or linings of thin plywood or Presdwood are most widely used. In general, the soft woods or woods of coniferous trees are used for form lumber because they are widely distributed, easy to work, are light in weight and hold nails well. Long leaf Southern yellow pine, Douglas fir which is sometimes called Oregon pine, West Coast hemlock, Northern White, Idaho White, Sugar and Ponderosa pine are all suitable woods. Norway pine and Eastern spruce of good quality

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‡By R. S. Phillips, Journal Amer. Concrete Institute, Feb. 1939; *Proceedings* Vol. 35, p. 277.

can also be used. Eastern hemlock is generally not considered suitable and California redwood, while used for structural concrete forms, will stain the concrete and, therefore, should not be used for architectural concrete.

No. 1 common boards are generally recommended, but No. 2 dimension may be used for studs and wales if the forms are to be used only once. Second and third grades should be used for sheathing only if impressions of knots and other flaws are desired for architectural reasons. If the finished surface is to be uniformly smooth with only slight impressions of joint lines and grain marking, the boards should be No. 1 dressed and matched uniformly sized. There is some tendency to warp and cup, even though the boards are oiled. This is more pronounced, the wider the boards. For average work, 6 in. wide boards are used. Eight and 10-in. boards make the joint lines more pronounced because of cupping.

Rough textured surfaces showing pronounced grain marking and joint lines are obtained with re-sawed, square-edged lumber. For only mild accentuation of joint lines, No. 1 boards surfaced one side and two edges are used with the rough side of the lumber as contact surface. For very rugged textures, neither side of the sheathing is surfaced. The grain may be accentuated by wetting the lumber before oiling, or more effectively by spraying it with ammonia.

In every load of lumber some of the boards are better than others. When unloading, the better boards should be set aside for the contact surface. They should be stockpiled by separating the layers with ¼-in. laths.

Where plywood, Presdwood or similar material is used as lining, the lumber used for backing need not be of as high grade as that used for contact surfaces. It need not be T and G material but must be sized to uniform thickness. Plywood ¼ in. and ⅜ in. thick is used for lining, but since it usually must be backed up solidly it is more economical to use thicker material without backing. The ⅝, 11/16 and ¾ in. thicknesses are most commonly used, nailed directly to the studs without backing. Only plywood made with a waterproof glue should be used. Douglas fir plywood made especially for formwork is now available and only the grade specifically intended for concrete forms should be used for that purpose.

Presdwood should be tempered, that is, treated to minimize absorption, swelling and buckling.

Wire ties have so many objections they cannot be considered for architectural concrete. Ties that are so adjustable in length as to

permit tightening of forms and that leave no metal closer than 1½ in. of the surface are most satisfactory. The hole left by the tie should not be larger than ⅞ in. in diameter and ties leaving even smaller holes are preferred.

TREATMENT OF MATERIALS BEFORE ASSEMBLING

As previously stated, where board sheathing is used for the contact surface, the boards should be sorted and the best ones selected for the face form. The selected sheathing should be stacked and protected from damage until ready to use.

Plywood should be oil treated at the plant and it should be re-oiled or lacquered on the job before it is used. This can be done on arrival at the job and then the sheets stacked until they are to be used.

Presdwood should contain some moisture when it is used. To assure this, the sheets should be thoroughly wetted with water on the screen side and stacked screen side to screen side for at least 12 hours. The contact surface of Presdwood should be oiled or lacquered the same as plywood.

ERECTION OF FORMS

Certain limiting factors in architectural concrete and experience on many buildings permit making recommendations for spacing of studs, wales and ties to prevent objectionable distortion of forms under most conditions. For buildings, the forms are usually erected in story heights or less. To minimize water rise and prevent sand streaking and other imperfections on the surface, the rate of placing should be limited to about two feet per hour.

Studs should be placed on not more than 16-in. centers when using 1-in. sheathing or 11/16-in. or thicker structural grade plywood. With ⅝-in. plywood, the studs should be spaced not more than 12 in. on centers. If the grain of the outer plies of plywood is parallel to the studs, the stud spacing should be reduced 2 in. for all thicknesses. Wales should be not more than 24 in. apart and ties not more than 27 in. These maximum spacings are based on using 2 x 4-in. studs and wales, the wales being doubled and the concrete placed at a rate not to exceed 2-ft. an hour. Joints in the top and bottom wales should be staggered at least the spacing of the form ties. At all external angles the wales should extend sufficiently so that they can be firmly locked to prevent movement and subsequent leakage at the corner. Vertical kick strips and wedges as shown in Fig. 1 should be provided. A somewhat similar condition requiring the use of long wedges and blocks to insure tight corner joints is shown in Fig. 2.

For lifts over 10 ft. high and at all spandrel beams regardless of the height of forms, double 2 x 6-in. vertical wales not more than 10 ft.

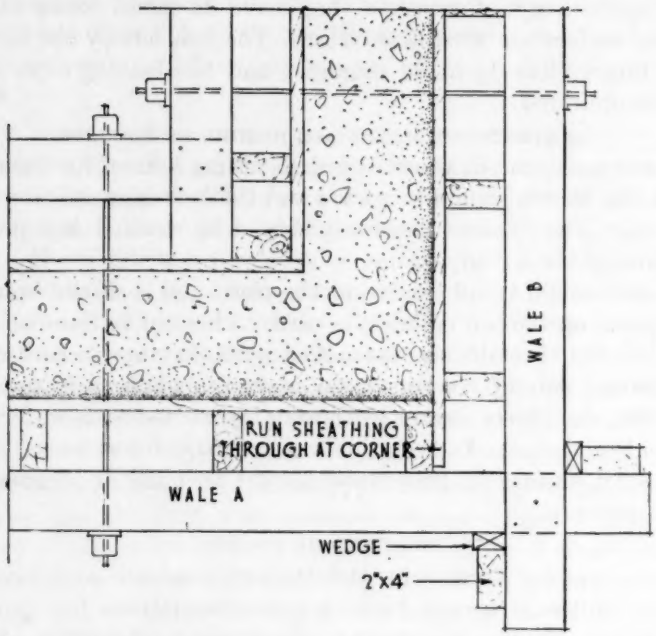


FIG. 1—DETAIL OF SHEATHING, STUDS AND WALES AT CORNER

apart and extending the full height of the forms should be bolted to every other set of horizontal wales. This precaution should maintain the forms in straight, true alignment. Fig. 3 shows good forming with vertical as well as horizontal wales.

Sheathing boards should be nailed at every stud with 6-d box nails, 6-in. material with two nails, 8-in. and 10-in. material with three nails—at both edges and at center. Joints should be exactly horizontal or vertical. Vertical joints should be not more than one board wide, staggered at least two feet and should be made only at a stud.

Plywood sheathing should have all joints solidly backed. The short horizontal 2 x 4's between the studs in Fig. 3 illustrate this requirement. Edges should be nailed with 6-d box nails not over 8 in. apart and nail spacing should be the same along all studs. Where ¼-in. and ⅜-in. plywood are used as lining, it should be nailed at 6 to 8-in. intervals along all four edges with 3-d blue shingle nails and in addition at least one nail to each square foot to prevent bulging. Rustication strips and strips for chamfered corners should be nailed with small brads at not more than 8-in. intervals along the edges to prevent leakage at these locations.

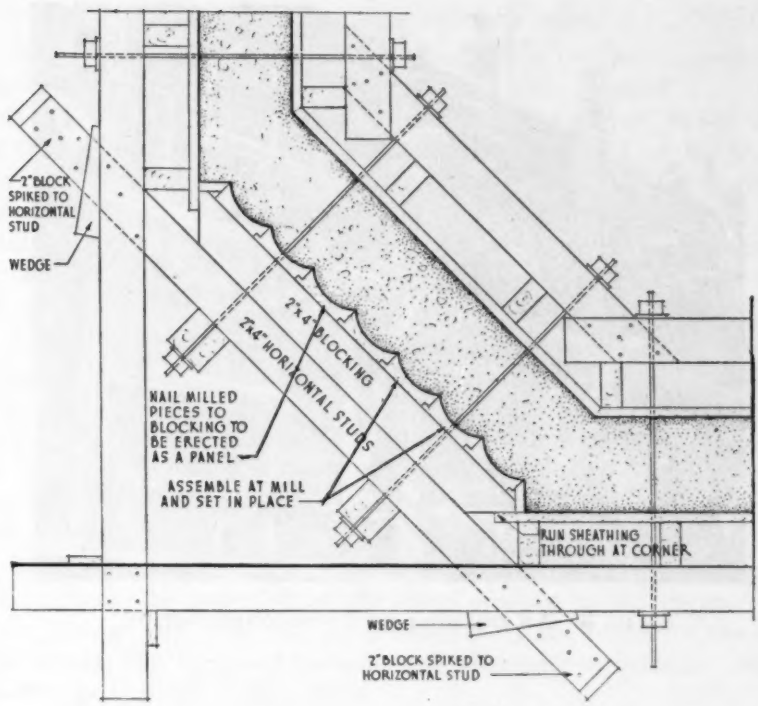

2"BLOCK SPIKED TO HORIZONTAL STUD
WEDGE
2"x4" BLOCKING
2"x4" HORIZONTAL STUDS
NAIL MILLED PIECES TO BLOCKING TO BE ERECTED AS A PANEL
ASSEMBLE AT MILL AND SET IN PLACE
RUN SHEATHING THROUGH AT CORNER
WEDGE
2"BLOCK SPIKED TO HORIZONTAL STUD

FIG. 2—DETAIL TO INSURE TIGHT FORM CORNERS

If the sheets of plywood sheathing are not of exactly the same thickness, small wedges should be driven between the thinner sheets and studs to bring them into a plane with the others.

Plaster waste molds should be nailed to the supporting timbers with common nails, countersunk and pointed with patching plaster. Where practicable, the back side of waste molds should be made flat to bear against the form sheathing or a framework of studs. Where necessary, wads of plaster with a liberal amount of jute may be used to block out waste molds so that they will bear against the form framing. Jute fiber dipped in plaster should be twisted about the studs and blocking to secure the mold in place. After the molds are secured in place, all joints in the molds and between molds and adjoining framework should be pointed with patching plaster. All new plaster should be given two coats of shellac to match the adjoining plaster. A plaster waste mold set in the forms is shown in Fig. 4.

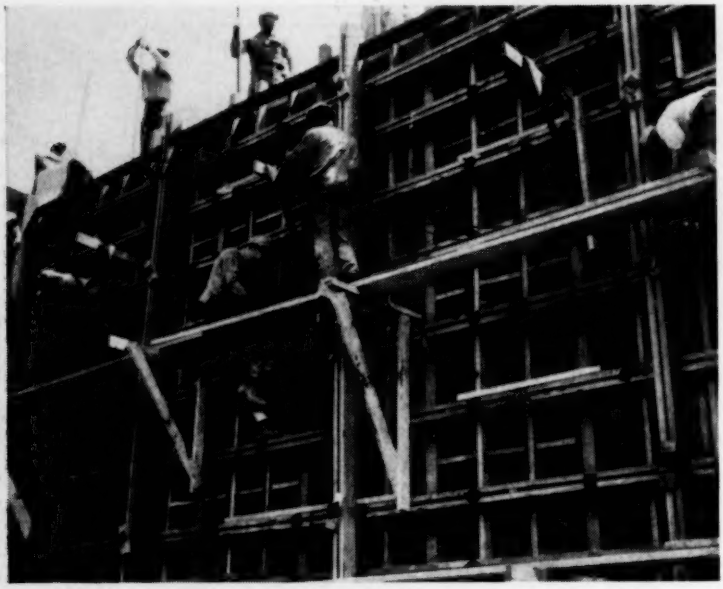


FIG. 3—AN EXAMPLE OF FORM CONSTRUCTION

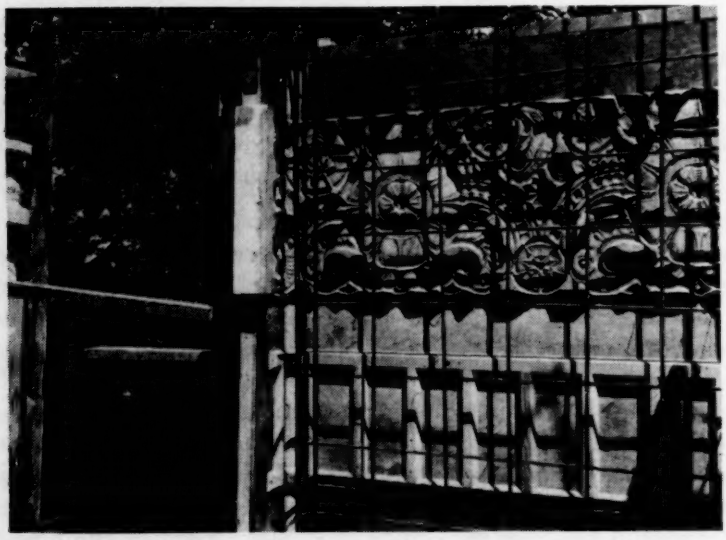


FIG. 4—A PLASTER WASTE MOLD SET IN THE FORM

PROVISION FOR CONSTRUCTION JOINTS

Where horizontal construction joints are to be placed, provision should be made so that the joint will be a level, straight line. One method is to stop the outer face form sheathing exactly at the level of the joint. Concrete should be placed to a point above the top of form, allowed to settle, any excess scraped off and the area between outer form and reinforcing steel screeded while the concrete is still plastic. The rest of the surface is wire brushed the following morning to remove laitance and expose the aggregate to insure good bond. This will give a straight line at the joint.

Another procedure is to nail a level strip of 1-in. square edged dressed board to the inside face of the form at the joint line as shown in Fig. 5. Concrete is placed to a level ½ in. or so above the underside of the strip. About an hour later the strip is removed and any irregularities in the joint line are leveled off with a screed. As in the first method the surface of the joint is wire brushed to remove laitance and expose the aggregate.

Not more than 3 or 4 in. below the construction joint, ⅝-in. stud bolts should be provided at the same spacing as the regular ties. The bolts at this level will permit tightening the forms for the next lift against the hardened concrete and thus prevent leakage. Fig. 5 shows the location of these bolts.

WINDOW AND DOOR OPENINGS

It is advisable to run the studs past window and door openings and the sheathing part way past the openings to provide a substantial backing to which the forms for the opening can be secured. The box forming the opening is best made of 2-in. material cleated with 2 x 4's at 24-in. intervals. Strips may be nailed to this to form the recess for window sash. Cross braces horizontally and vertically should be located at each cleat or an inner frame of 2 x 4's should be provided.

It is usually necessary to get at the sill to work the concrete into place and finish it. The sill of the form should therefore, be omitted or made in two sections for easy removal. At door openings with deep reveals, tie rods must be placed as close to the corner as possible to prevent spreading of the forms at the return. Wales should be well blocked and secured on the outside of the corner to prevent movement.

SPANDRELS

Where spandrels project above and below the floor it is desirable to provide a construction joint at the floor line. This can be done when the part of the spandrel below the floor line is sufficient for

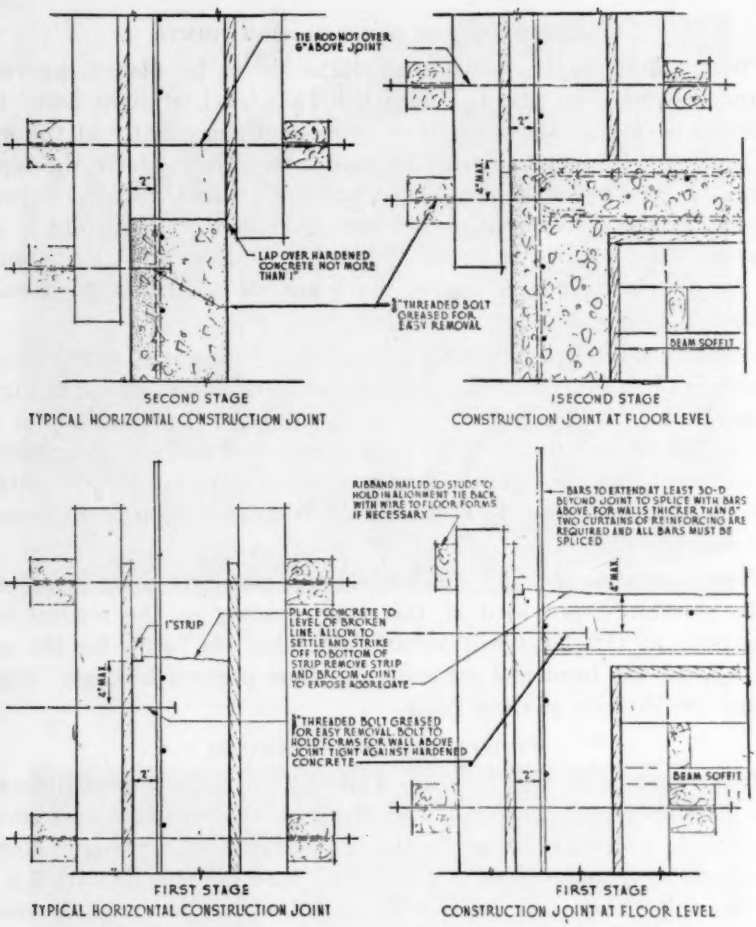

TIE ROD NOT OVER 6" ABOVE JOINT
LAP OVER HARDENED CONCRETE NOT MORE THAN 1"
3/4" THREADED BOLT GREASED FOR EASY REMOVAL
BEAM SOFFIT
SECOND STAGE
TYPICAL HORIZONTAL CONSTRUCTION JOINT
SECOND STAGE
CONSTRUCTION JOINT AT FLOOR LEVEL
RIBBAND NAILED TO STUDS TO HOLD IN ALIGNMENT TIE BACK WITH WIRE TO FLOOR FORMS IF NECESSARY
BARS TO EXTEND AT LEAST 30-D BEYOND JOINT TO SPLICE WITH BARS ABOVE. FOR WALLS THICKER THAN 8" TWO CURTAINS OF REINFORCING ARE REQUIRED AND ALL BARS MUST BE SPLICED
1" STRIP
PLACE CONCRETE TO LEVEL OF BROKEN LINE. ALLOW TO SETTLE AND STRIKE OFF TO BOTTOM OF STRIP REMOVE STRIP AND BROOM JOINT TO EXPOSE AGGREGATE
3/4" THREADED BOLT GREASED FOR EASY REMOVAL. BOLT TO HOLD FORMS FOR WALL ABOVE JOINT TIGHT AGAINST HARDENED CONCRETE
BEAM SOFFIT
FIRST STAGE
TYPICAL HORIZONTAL CONSTRUCTION JOINT
FIRST STAGE
CONSTRUCTION JOINT AT FLOOR LEVEL
HORIZONTAL CONSTRUCTION JOINTS

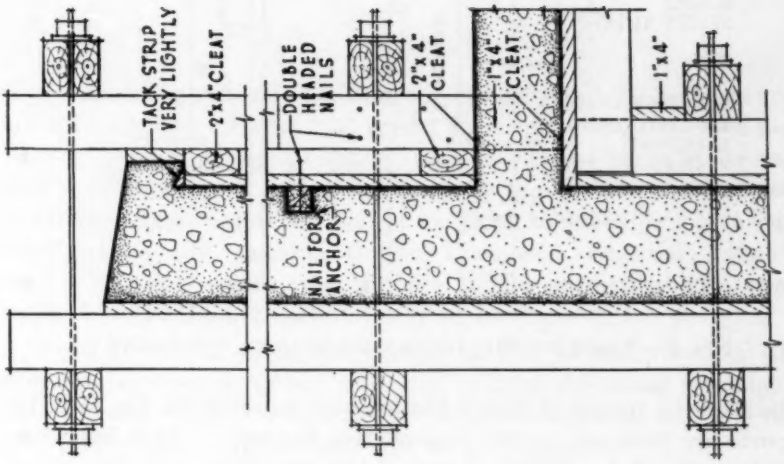
FIG. 5—DETAILS OF FORM CONSTRUCTION AND TIES

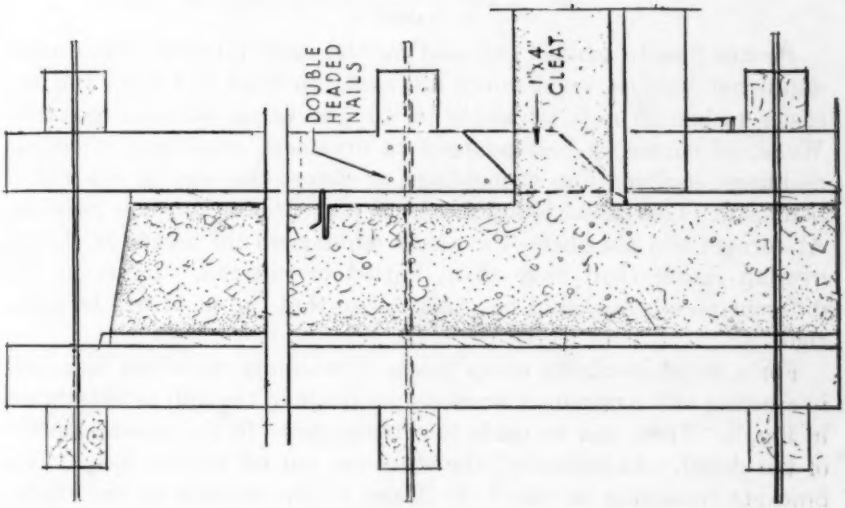
structural purposes. The form is then constructed to the floor line. The forms for the upper part of the spandrel and the rest of the wall are erected after the lower section and the floor have been concreted. For completely upturned spandrels, the forms should be erected so that the adjoining floor slab, beams and spandrel can be concreted in a continuous operation

The forms for parapets are built much the same as for spandrels. Usually panels 10 or 12 ft. long are used for the inside of the parapet wall. These are supported on the roof slab form by 1 x 4 pieces nailed

FIG 6 (LEFT)—SEE TEXT

FIG. 7 (RIGHT)—SEE TEXT





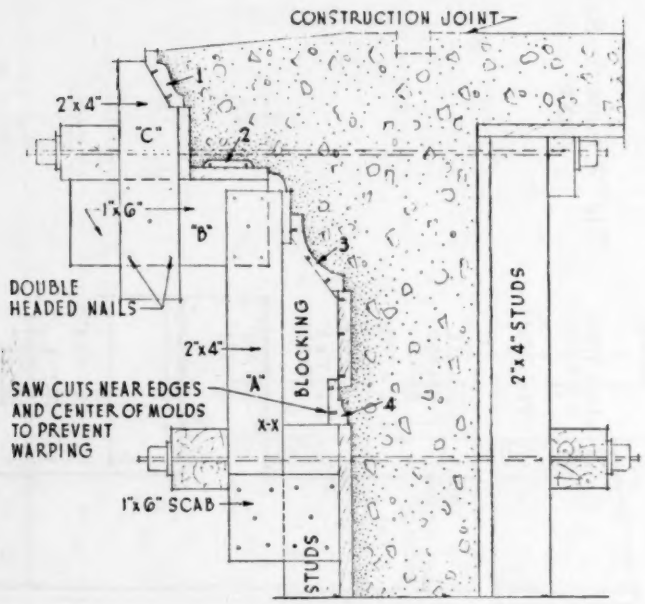


FIG. 8—FORM DETAIL, INVOLVING MOLDED ORNAMENT

to the studs by means of double headed nails as shown in Fig. 6. The supports are removed before the concrete hardens so that holes are not left in the slab. Patented types of raggle strips are often called for. In other cases wood strips may be used as illustrated in Fig. 7.

ORNAMENT

Plaster "waste molds" are used for the more intricate ornamental detail but because wood molds are easier to erect and strip and are easier to handle it is advisable to use the wood wherever possible. Wood, of course, is best adapted to ornament consisting of simple moldings, combinations of moldings, or shapes that can be cut with a band saw. Leakage at joints should be prevented as much as possible. At corners and elsewhere, the pieces making up the assembly should overlap rather than have them butted or mitered. Joints in the different members should be staggered so that the mold will be more rigid.

For a detail involving many pieces of moldings, time can be saved in erecting and stripping if brackets are made in the mill as illustrated in Fig. 8. These can be made to a template to fit the general profile of the detail. As indicated, the studs are cut off at line X-X. The brackets consisting of pieces A, B and C are scabbed to the studs.

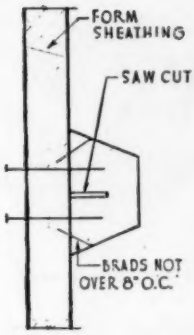


FIG. 9—BEVELED STRIPS PROVIDE RECESSES

The wales are put in place to hold the brackets and the mold members are then applied. Pieces 1, 2, 3 and 4 are moldings, the other pieces ripped to size from stock lumber. Saw-cuts in the backs of the moldings prevent warping and wedging of the lumber which might result in broken edges. Material that is too thick or too wide may swell or warp. Generally, nothing thicker than 2-in. material should be used. When necessary, the member should be divided into two or more pieces so thinner material may be used.

Wood molds or strips for recesses should be made with a slight draw or bevel so that they will strip without damaging the edges of the concrete. Wide saw-cuts in the backs of the pieces will also make stripping easier. Fig. 9 illustrates these suggestions.

Discussion, to close in April, 1940 JOURNAL should reach A. C. I. Secretary in triplicate by Feb. 1, 1940.

Vol. 36 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN *APRIL 1940*

Discussion of a paper by A. J. Boase

Forms for Architectural Concrete*

BY E. P. PITMAN AND THE AUTHOR

BY MR. PITMAN†

Mr. Boase's paper covers very thoroughly the subject of forms for architectural concrete; however, there are three points on which discussion may be made in light of the results obtained on a job recently completed in New York City.

Under the heading "Treatment of Materials before Assembling," it is stated that "plywood should be oil treated at the plant and it should be re-oiled or lacquered on the job before it is used." Information from manufacturers of plywood for forms and of lacquers for coating forms indicates that lacquers, if used, should be applied to bare plywood. It is questionable if lacquer should be used at all. On the above mentioned job certain areas of the forms were lacquered plywood, and other areas were oiled square-edged sheathing. Practically all the concrete which was placed against lacquered forms has a very smooth glassy surface, but is unsightly, due to blotches of light and dark spots; on the other hand, the concrete placed against oiled forms on the back of the same wall is uniform in appearance.

The discolorations in the concrete placed against lacquered forms are not surface stains, but extend through the surface of the concrete. The spots are not removed by washing with a ten-per cent solution of muriatic acid, nor by washing with the solvent used in the manufacture of the lacquer. The surface of the concrete was cut at various points with a chisel, and the concrete at the bottom of the cuts in both the light and dark areas appeared the same.

*JOURNAL, Amer. Concrete Inst., Nov. 1939; *Proceedings* Vol. 36, p. 193.
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Such spots are possibly due to varying water content of parts of the same batch of concrete or of different batches. It is felt that the difference in the absorption of forms in the two faces of the wall would account for the difference in appearance of the concrete.

A typical example of variation in surface color may be seen in Fig. 1 of a pamphlet published by the Portland Cement Association, entitled "Forms for Architectural Concrete."

Since the primary reason for the use of lacquer is to permit the contractor to re-use his plywood forms several times, and since architectural concrete costs are much higher than those for ordinary concrete, and considering also the small cost of plywood compared to the cost of the concrete, serious thought should be given to the use of a coating which permits slight absorption into the form, even if by doing so the re-use of plywood forms is prevented.

The design of the concrete mixture on the job before referred to was made in accordance with recommendations of the Portland Cement Association, which recommendations had been incorporated into the specifications. The following data will indicate the character of the concrete: Proportions, 1:2.05:2.90 (dry and rodded volumes); 5.86 gallons of water per sack of cement; 6.20 sacks of cement per cubic yard of concrete; 3 inch slump. The cement, sand and 1½ inch trap rock were measured by weight at a batching plant located away from the job. The mixing was done in truck mixers; the water, however, was not added to the other ingredients of the concrete until after the truck mixers reached the job.

Under the heading "Erection of Forms," Mr. Boase calls for the rate of placing to be limited to about two feet per hour. It should be brought out that the placing should be continuous at the above rate. In order to carry out this requirement, it may be necessary to limit the volume of each concrete batch, even if this requires the mixing of many small batches in order to place concrete continuously. In one instance where the volume of the batch was not limited, the height of one hour's placing was reached in less than one hour, and no additional concrete was placed until the start of the next hour. The concrete surface shows horizontal lines at intervals approximating an hour limit of placing of concrete.

Under the same heading mention is made of driving wedges between sheets of plywood and the studs in order to have the adjacent forms in the same plane. However, even if the adjacent panels are in the same plane it is practically impossible to prevent the joint from showing in the concrete. An example of joints in forms showing on the surface of

the concrete is shown in Fig. 4(a) of the Portland Cement Association pamphlet mentioned above. The marks of joints vary, some are practically invisible and others are visible at a long distance. The non-uniformity of joint marks does not present a pleasing appearance, since frequently two pronounced joints will be next to each other, and then a group of several very faint joints will occur. It is felt that rustication strips should be placed at each form joint and that sufficient other rustication strips should be introduced so that all the paneling forms a pattern which will be in harmony with the architectural features of the structure. These rustication strips need not be large—a strip having a face of one-quarter inch would be sufficient —and they would produce uniform joints throughout the structure.

These comments on Mr. Boase's paper are not intended to be critical or to indicate an antagonism to architectural concrete, but merely recount some of the difficulties encountered on a job in the New York City area; it is felt that only by becoming aware of some of the possible pitfalls can the results obtained through the use of architectural concrete be improved.

AUTHORS' CLOSURE

Mr. Pitman's experience with lacquered plywood is so contrary to the experiences of others that it raises some question as to the type of lacquer used and the method of its application. On most jobs the lacquer has been thinned out with the proper solvent and applied by brush in a single coat before using the plywood and again before each reuse but after cleaning the plywood very thoroughly. The process does not completely seal the surface but no doubt does reduce absorption considerably. Observations have been made on many jobs and the results from the standpoint of appearance have been almost invariably favorable. On one large building where the plywood panels were oiled for the first story and lacquered for the upper stories, there is noticeable difference in color, the concrete in contact with the lacquered forms being much lighter and more uniform. The accompanying photograph (Fig. 10) shows this difference. The illustration shows forms in place for the third story and the scaffolding.

In Fig. 1 of the booklet "Forms for Architectural Concrete," published by the Portland Cement Association and cited by Mr. Pitman, the forms were not lacquered but were oiled so the slight variation in color cannot be attributed to a lacquer form treatment.

The primary reason for the use of lacquer is not to permit reuse of the form as much as it is to secure a pleasing surface. The main difference in cost, however, between architectural concrete and other

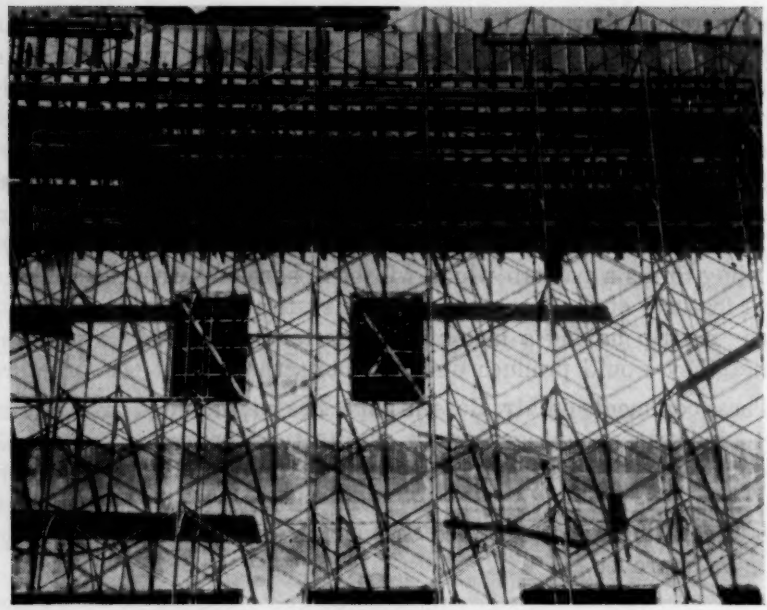

FIG. 10—PLYWOOD PANELS, OILED FOR THE FIRST STORY, LACQUERED ABOVE, GIVE THE RESULTS SHOWN.

concrete is in the forms and any reduction in the cost of forms tends to reduce the spread.

Mr. Pitman's suggestion that concrete placing be continuous is excellent. Placing continuously at a rate of about two feet or less per hour will reduce the tendency of sand streaking. The interval between successive layers should be short enough to avoid horizontal lines appearing on the surface.

Rustication strips at the form joints have been very successfully used and in many cases can be fitted into the architectural design. With careful workmanship, however, using small wedges to bring the plywood panels into line and pointing the joints between the panels to prevent leakage and the subsequent discoloration, excellent results have been secured on many jobs without rustication strips.

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Prestressed Reinforced Joists Under Loading Tests*

By R. E. Mills†
MEMBER AMERICAN CONCRETE INSTITUTE

AND W. B. Miller‡

SYNOPSIS

Presented in this paper are data and conclusions from a series of studies made at the Materials Testing Laboratory, Purdue University, which were designed to evaluate several possibilities in the development of small light precast beams or joists for small house construction. Data herein recorded cover the design and testing of three typical precast joists (3 in. by 8 in. by 12 ft. long) as follows: a prestressed haydite concrete joist, a conventional haydite joist, and a prestressed segmental Rostone joist. Comparative test values were determined for both conventionally-reinforced and prestressed units and are presented in the accompanying charts. The results given include a theoretical analysis of each joist in comparison with the stress measurements in the tests.

INTRODUCTION

In adapting reinforced concrete to an ever-increasing field of usefulness, the problem of economical utilization of the strength of both the steel and the concrete is of utmost importance. Under conventional methods of design and construction, reinforcing steel in compression is usually understressed; while in tension it is customary to assume that the steel carries all of the stress. As a result of this condition, tensile stresses in concrete often reach the ultimate and fissures or small cracks appear. While cracks of this nature may have no immediate structural significance, nevertheless they indicate a lack of homogeneity in the action of conventionally reinforced concrete.

The development of practical methods of prestressing the steel and the concrete affords an opportunity of utilizing to advantage the

*Received by the Institute June 30, 1939
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difference in the characteristics of the two materials. Several ingenious methods of prestressing have been devised and used to a limited extent in both the United States and Europe.[1]

The scheme of destroying the bond between the steel rods and the concrete, with subsequent prestressing of the rods in tension and the concrete in compression, has been investigated at the Materials Testing Laboratory, Purdue University. The first series of tests was conducted during 1933-34 in which the principle of prestressing was investigated in reinforced concrete pressure pipes of 30-, 42-, and 60-in. diameters[2]. A second series of tests was conducted during 1934-35 on large concrete beams, eight inches by 14 in. by 13.5 ft. long. Comparative values were determined for both conventionally reinforced and prestressed units[3].

Since that time, additional tests have been under way attempting to develop a small light precast beam or joist for small house construction. Data herein presented represent test measurements for three typical precast joists, 3 by 8 in. by 12 ft. long. See Fig. 1 and 2.

GENERAL SCHEME OF JOIST CONSTRUCTION

The general schemes of construction of these joists were as follows:

a—Prestressed haydite concrete joist. A detail plan of the construction of this joist is shown in Fig. 1. The $\frac{3}{8}$ in. round rod in the top of the joist was bonded to the concrete, while the bond of the $\frac{3}{4}$ in. rod in the bottom was destroyed by a specially prepared coating material. The bottom or tension rod was provided with threaded ends, nuts, and bearing plates, to effect prestressing.

b—Conventional haydite concrete joist. The construction of this joist was a duplication of the prestressed joist described in (a), except that the bottom or tension rod had hooked ends and was bonded to the concrete. For the purpose of a comparison, stirrups were omitted from this joist as well as from joist (a).

c—Prestressed segmental Rostone joist. This joist was constructed of precast segments of a synthetic stone known as Rostone. Each segment was three inches wide by eight inches deep by approximately two and one-half feet long with proper longitudinal holes provided in both top and bottom through which the steel reinforcing rods were threaded. The dimensions of the completed joist as well as the relative locations of the steel reinforcing rods were the same as those shown in Fig. 1 for the prestressed concrete joist. The end bearing plates extended for the full height of the joist thus providing bearing for

[1]Prestressed Reinforced Concrete and its Possibilities for Bridge Construction, by Ivan A. Rosov; Transactions of the Am. Soc. C. E., Vol. 103, p. 1334, 1938

[2]An Investigation of Stresses in Prestressed Reinforced Concrete Pipes, by Ray B. Crepps; Bulletin No. 46, Engineering Experiment Station, Purdue University

[3]Concrete Beam with Prestressed Bars Tested, by W. K. Hatt; Engineering News-Record, Sep. 13, 1934

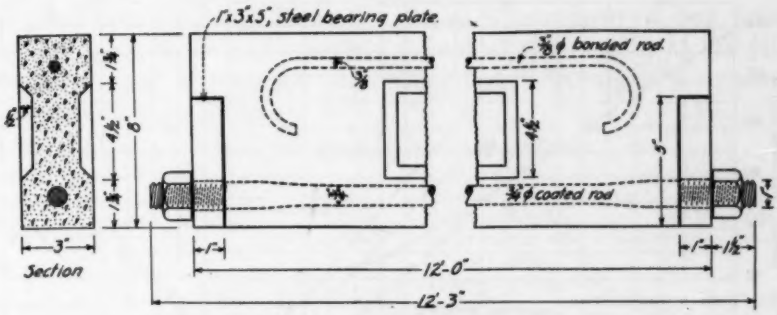

FIG. 1—DETAIL OF 3 IN. X 8 IN. X 12 FT.-0 IN. HAYDITE CONCRETE JOIST

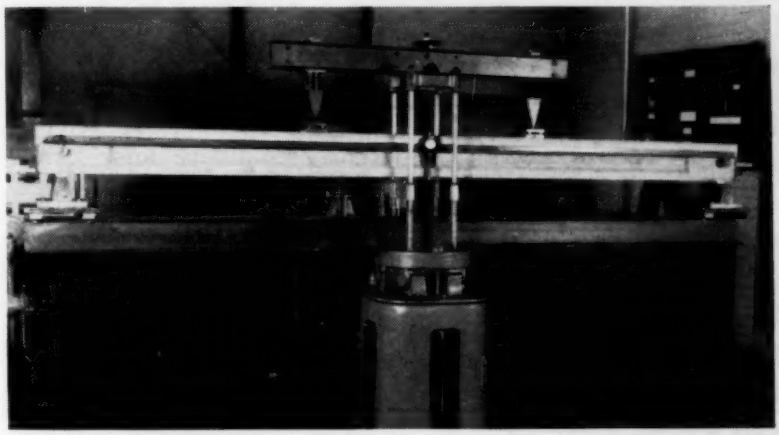

FIG. 2—PRESTRESSED HAYDITE CONCRETE JOIST UNDER LOADING TEST

both top and bottom rods which had threaded ends and nuts. The cross section of the joist was rectangular throughout its length with no paneling along the sides. In assembling the units of the joist, cement mortar joints were provided between the several segments, and between the end segments and the bearing plates.

METHOD OF PRESTRESSING

Prestressing of the joist was accomplished by tightening the nuts on the bottom or tension rod (see Fig. 1). This induced initial compression stresses in the lower part of the joist which were balanced by tensile stresses in the upper part of the joist and in the bonded rod. In the case of the segmental joist the tension in the upper rod was adjusted so that small compression stresses existed in the top of the joist during prestressing, thus preventing the opening of the mortar bearing joints at the top. In each case the prestress in the joist was

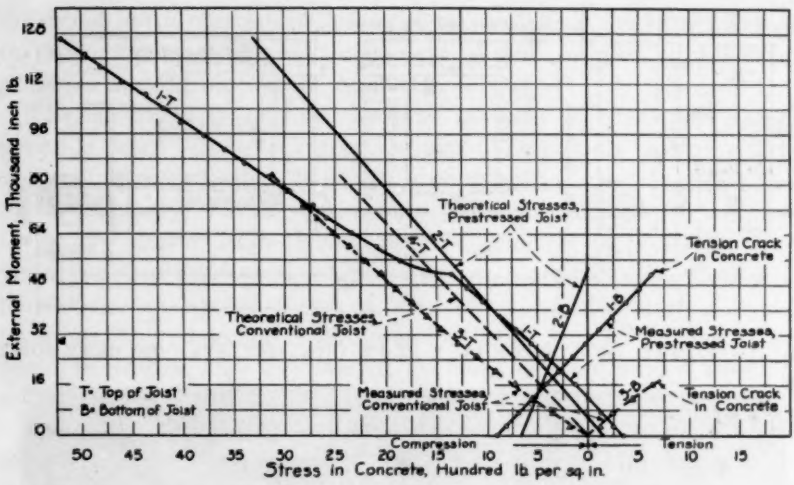

FIG. 3—COMPARISON OF MEASURED AND THEORETICAL CONCRETE STRESSES FOR A PRESTRESSED HAYDITE CONCRETE JOIST AND A CONVENTIONAL HAYDITE CONCRETE JOIST

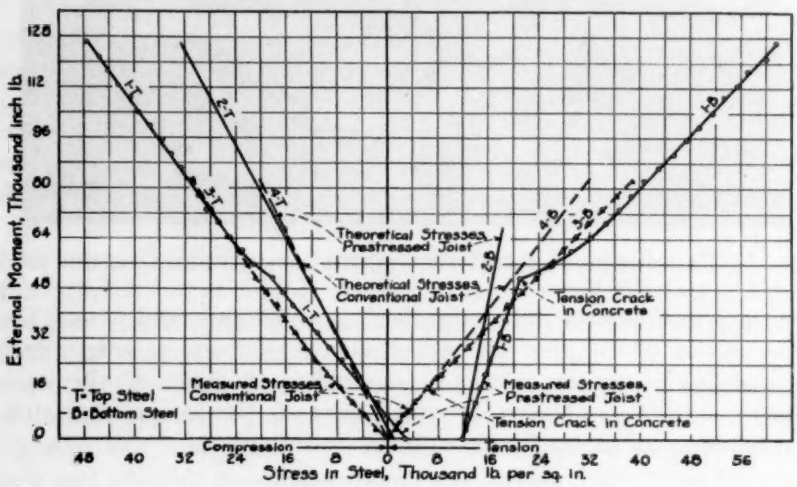

FIG. 4—COMPARISON OF MEASURED AND THEORETICAL STEEL STRESSES FOR A PRESTRESSED HAYDITE CONCRETE JOIST AND A CONVENTIONAL HAYDITE CONCRETE JOIST

adjusted to a predetermined amount at the time of the loading test. The amount of this prestress is shown on the accompanying charts.

LOADING TEST PROCEDURE

The loading tests were conducted in a Universal testing machine of 30,000 pounds capacity (See Fig. 2). The loads were applied at

the third points on a 11-ft. span. For each increment of load, stress measurements were made at the top and at the bottom of the joist and in the steel, at midspan. Corresponding deflection measurements were also made at midspan.

DATA PERTINENT TO TEST JOISTS

General:
Length of joist.....12 ft.
Length of tested span.....11 ft.
Loading.....third points
Steel:
E_s.....30,000,000 p.s.i.
Elastic limit.....50,000 p.s.i.
Haydite Concrete Joist:
Cement—tensile strength at 28 days..445 p.s.i.
Concrete—haydite aggregate, 1:2½ mix, compressive strength at 28 days..6,520 p.s.i.
E_c.....2,267,900 p.s.i.
Weight of joist.....15½ lb. per running ft.
Weight of concrete.....97 lb. per cu. ft.
Age of joist at time of test—44 days (28 days moist curing).
Segmental Rostone Joist:
Material.....Rostone, a synthetic stone
Compressive strength.....7,900 p.s.i.
E_s.....2,750,000 p.s.i.
Weight of joist.....20.6 lb. per running ft.
Weight of Rostone.....124.0 lb. per cu. ft.

DISCUSSION OF RESULTS

The results of the stress measurements for the loading tests on the haydite concrete joists are shown in Fig. 3 and 4 while those for the segmental Rostone joist are shown in Fig. 5 and 6. Corresponding values of deflection for all joists are grouped in Fig. 7. Unit stress values at the top and at the bottom of the joist and in the steel are reported as: (1) measured in a tested prestressed joist; (2)* computed by theory for the prestressed joist; (3) measured in a tested conventional joist; and (4)** computed by theory for the conventional joist. For comparison, see typical data in Tables 1, 2 and 3.

A perusal of the several charts shows the theoretical stresses to be reasonably well in agreement with the measured stresses, within the limits of working stress values. Variations between the theoretical and measured stresses are approximately of the same order for both the prestressed and conventional joist.

The prestressing of a joist first produced an upward bending as shown by the negative deflections in Fig. 7. Under subsequent loading, an external moment of 18,400 in. lb. was required to deflect the haydite concrete joist to its normal form. A corresponding value for the segmental stone joist was 11,600 in lb. The deflection of the conventional joist under a moment of 18,400 in. lb. was 0.147 in. When the external moment was increased to 51,300 in. lb., the following order of deflections were recorded: Prestressed haydite concrete joist 0.211 inches; prestressed segmental stone joist 0.171 in.; and conventional haydite concrete joist 0.410 in.

Loading tests were continued on each joist to the point of final failure, and a complete record of the data is shown in the several

*Theory of prestressed rectangular beams by Ivan A. Rosov; Transactions of the Am. Soc. C. E., Vol. 103, p. 1341, 1938

**Theory of the flexure of beams by Turneaure and Maurer, Principles of Reinforced Concrete Construction, fourth edition, page 30.

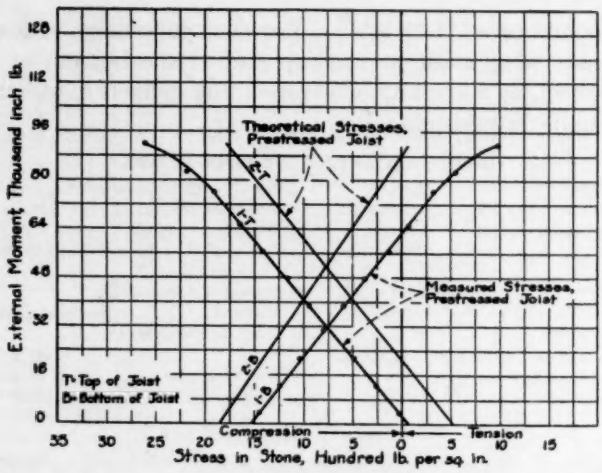

FIG. 5—COMPARISON OF MEASURED AND THEORETICAL STONE STRESSES FOR A PRESTRESSED SEGMENTAL ROSTONE JOIST

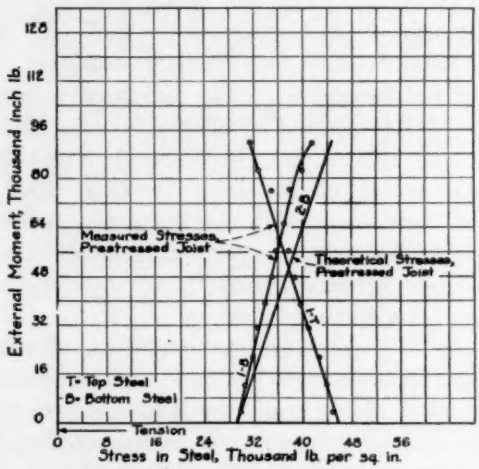

FIG. 6—COMPARISON OF MEASURED AND THEORETICAL STEEL STRESSES FOR A PRESTRESSED SEGMENTAL ROSTONE JOIST

charts. Final failure in each prestressed joist developed on the compression face within the middle third of the span, while the conventional joist failed in shear near one end support. It is interesting to note from Fig. 3, 4, and 7, that following the occurrence of a tension crack in the prestressed haydite concrete joist, both the steel and concrete stresses, as well as the deflections, approach the corresponding observed values for the conventional joist.

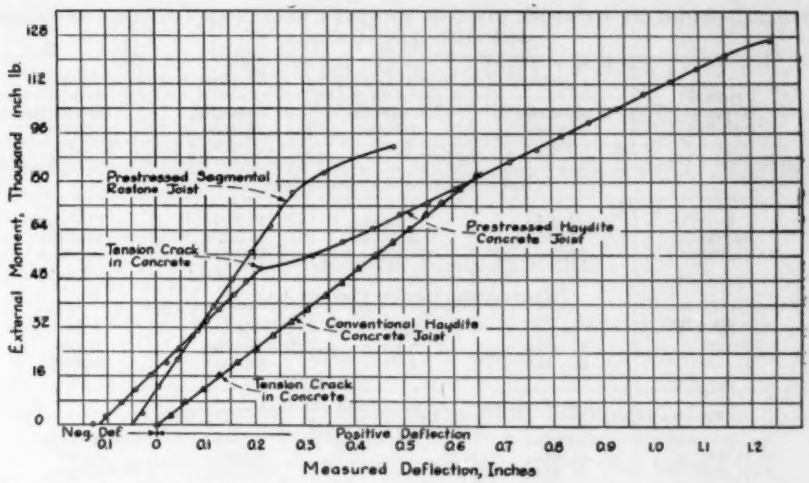

FIG. 7—COMPARISON OF DEFLECTIONS FOR PRESTRESSED AND CONVENTIONAL JOISTS

TABLE 1—SUMMARY OF CONCRETE STRESSES AND STONE STRESSES SHOWING RELATIONS BETWEEN MEASURED AND THEORETICAL VALUES

External Moment inch lb.	Top of Joist		Bottom of Joist	
	Measured Stress p.s.i.	Theoretical Stress p.s.i.	Measured Stress p.s.i.	Theoretical Stress p.s.i.
Prestressed Haydite Concrete Joist				
0	340-t	150-t	900-c	670-c
24,900	410-c	530-c	150-c	350-c
51,300	1,350-c	1,250-c	650-t	10-c
Conventional Haydite Concrete Joist				
0	0	0	0	——
24,900	1,120-c	730-c	cracked	——
51,300	2,050-c	1,520-c	cracked	——
Prestressed Segmental Rostone Joist				
0	70-t	510-t	1,530-c	1,850-c
24,900	580-c	110-c	930-c	1,330-c
51,300	1,280-c	770-c	280-c	780-c

t = Tensile Stress c = Compressive Stress

CONCLUSIONS

1. Prestressing of a small precast member such as a joist affords an opportunity to develop a new field for reinforced concrete. It also makes possible the assembling of segments or small units into beam or joist construction as shown by the segmental stone joist tests.

2. Prestressing of precast members will permit handling of the units with less danger of damage.

TABLE 2—SUMMARY OF STEEL STRESSES SHOWING RELATIONS BETWEEN MEASURED AND THEORETICAL VALUES

External Moment inch lb.	Steel, Top of Joist		Steel, Bottom of Joist	
	Measured Stress p.s.i.	Theoretical Stress p.s.i.	Measured Stress p.s.i.	Theoretical Stress p.s.i.
Prestressed Haydite Concrete Joist				
0	2,800-t	800-t	11,700-t	11,700-t
24,900	7,300-c	5,900-c	16,300-t	14,200-t
51,300	18,200-c	12,800-c	21,100-t	16,800-t
Conventional Haydite Concrete Joist				
0	0	0	0	0
24,900	11,300-c	6,100-c	10,600-t	9,600-t
51,300	20,700-c	12,600-c	23,600-t	19,800-t
Prestressed Segmental Rostone Joist				
0	45,700-t	——	29,200-t	29,200-t
24,900	41,900-t	——	32,200-t	33,400-t
51,300	37,700-t	——	35,400-t	37,800-t

t = Tensile Stress c = Compressive Stress

TABLE 3—SUMMARY OF DEFLECTION MEASUREMENTS SHOWING RELATION BETWEEN PRESTRESSED AND CONVENTIONAL JOISTS

External Moment inch lb.	Deflection Measurements, Inches		
	Prestressed Concrete Joist	Conventional Concrete Joist	Prestressed Segmental Stone Joist
0	− 0.129	0.000	− 0.050
24,900	+ 0.048	+ 0.202	+ 0.057
51,300	+ 0.211	+ 0.410	+ 0.171

− = Negative Deflection + = Positive Deflection

3. Prestressing affords a great advantage in providing for a balanced strength relation between concrete and steel which should result in a saving of materials and permit increased span lengths.

4. Tensile stresses may be eliminated from the concrete in beam or joist members by proper prestressing, thus utilizing the strength of the entire cross-section of the unit and postponing the occurrence of cracks.

5. Steel possessing a high proportional elastic limit can be used to full advantage in a prestressed beam or joist.

6. Prestressing of a joist appears to stiffen the member as evidenced by the slope of the load deflection curves in Fig. 7.

Discussion, to close in April, 1940 JOURNAL, *should reach A. C. I. Secretary in triplicate by Feb. 1, 1940.*

Vol. 36 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

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Job Problems and Practice

Some will ask questions — Some will answer them — Some will do both

A. C. I. members are invited to use this new JPP department as an informal means toward mutual help. When a problem holds possibilities of general interest the discussion will be briefed in these pages.

If you have a problem—present it; a question, ask it. If you know the answer to a question asked, or if you can contribute something which may help in the solution, or have reason to disagree with the answers or suggestions published, your contribution will be welcome.

If you know of an interesting problem whose solution has already smoothed the way of someone in the field, tell us about it—some other A. C. I. member may need just that information.

The "answers" are the answers of individuals to whom the questions are referred and not of the A. C. I. as an organization.—EDITOR

*Handling materials and products in a small block plant?** (36-47)

Q—In small concrete block plants, such as the one I operate (especially in those not using steam curing) it is difficult to keep green block damp and free from drying air-currents. There is also the problem of raising the aggregate to mixer level. The expense of an overhead bin and elevating equipment seems prohibitive, and even with steam curing, which our plant has, it is hard to keep air currents away from the block.

These two problems of avoiding air currents in curing and raising materials suggest the possible advantage of a block plant under ground. Why not excavate for the shed and curing rooms as for a cellar with the floor level 7 ft. below grade? The mixer could be elevated 3 ft. above floor discharging onto a mortar box almost at level of block machine, requiring little lifting; or with more elevation directly into a

*The inquiry is from an A. C. I. member manufacturing concrete products in an eastern state. The answer is from an A. C. I. member who has had reason to give special study to products plant layout and operation.—EDITOR

hopper feeding the machine, eliminating the elevator and feeder. With a mixer standing 4 ft. high the side of the mixer would still be at ground level. Wheelbarrows could be loaded at ground level from stock piles of aggregate instead of on a raised wooden ramp which is inconvenient. Cement would also be on the mixer level, or higher if desired, as if unloaded from a truck to platform on the same level as the truck body it would be several feet higher than the top of the mixer. The curing rooms would be damp and with little fluctuation in temperature and with few air currents. I have never heard of such a block plant and would be glad to hear criticism of the idea.

A—It is evident that in the steam curing the inquirer is not providing enough additional moisture to prevent the block from drying out. This trouble would be largely overcome if he installed equipment to provide moisture independent from his steam line.

We believe it would be uneconomical to build the block plant in the basement and move unfinished products in an elevator to stock pile level. We know of no products plants constructed in this manner. We believe that it would be much more economical to set the mixer in a pit and use a skip hoist to carry the mixed concrete to a hopper above the block machines. This would overcome the principal objections to elevating the aggregate. The aggregate could be dumped as suggested on the platform at the same level with the truck body and from this platform be dumped directly into a batching car which in turn would be dumped directly into the mixer. Mixers ordinarily discharge from the bottom directly on to a skip car which is sufficient to hold the capacity of the mixer.

Such practice has been adopted by some of the newer plants in so far as the location of the mixer and skip hoist is concerned. Even though they have overhead bins and elevating equipment it reduces the height to which aggregates must be raised. It also allows for construction of a building with considerably less height.

Reference is made also to the report of A. C. I. Committee 708: Proposed Recommended Practice for the Manufacture of Concrete Building Block and Tile, A. C. I. JOURNAL, Apr. 1931.

*Floors in light occupancy buildings** (36-48)

Q—In reviewing the report of the Committee 804, Concrete Wearing Surfaces in Floors, and the comments in the recent issue of the JOURNAL (June), I find that the subject matter deals primarily with that type of surfacing used in factories, mills, etc. where large areas

*A letter from an Institute member.

are involved and power floats and other mechanical devices can be used to great advantage.

A serious problem prevails in placing concrete surfaces of small areas in buildings used primarily for light occupancy such as hospitals, office buildings, etc. The difficulty has been that contractors frown upon two-course construction on a hardened base and, in many cases, will not accept the responsibility of obtaining a proper installation.

It is of course understood that if the concrete surfacing is applied as recommended by the committee, it will serve equally as well for light occupancy buildings as for commercial or industrial buildings. Yet, with practically similar specifications, it has been recently experienced in a large construction program involving hundreds of buildings such as schools, hospitals, municiapl buildings, etc. that a great deal of the finished floor either had to be entirely torn up and replaced or patched. Even on those floors where no additional work was required, considerable concern is had with regard to its future behavior. In most cases the cement finishing was done by the general contractor and not by specialists in this field. Some contractors and even architects and engineers have gone so far as definitely to state that it is absolutely impossible to place a finished concrete wearing surface on a hardened base.

Some of the architects, engineers and inspectors have required the contractor to place the topping in alternate blocks. The topping laid on this basis has not given any appreciably better results than when laid continuously and subsequently cut by trowel into blocks.

This raises the question, should the topping be cut up into blocks if the hardened base was placed continuously without joints. If not cut up into blocks, will the difference in richness between a 2500 lb. base and a 1:2 topping that has no temperature reinforcing steel incorporated in it cause serious cracking and warping if otherwise placed as recommended?

Another question arises as to whether special precautions are needed when the topping is placed on a hardened base consisting of a floor slab of 2½ in. supported on concrete or steel joists. Moreover, will the possible uneven drying out on the underside of such slabs cause cracking and warping of the finish course?

Effect of animal and vegetable oils on concrete? (36-49)

Q—Does concrete resist the action of vegetable and animal fats containing free fatty acids?

A—The following information is taken from the chapter on the Resistance of Concretes to Various Organic and Inorganic Agents in

the book, "The Chemistry of Cement and Concrete," by F. M. Lea and C. H. Desch, published by Longmans, Green & Co., New York.

Vegetable and animal oils and fats are natural products composed of glycerides and esters, but they also contain some of the corresponding free fatty acids and alcohols. When such oils are brought into contact with concrete they react with the calcium hydroxide present in the hardened concrete, forming a calcium salt of the fatty acid.

The extent of the oil attack is dependent on the ease with which it can penetrate the concrete. Thus the viscosity of the oil, as well as the porosity of the concrete, is an important factor. Oils which have been exposed and have accumulated moisture and undergone oxidation become more active in attack. Disintegration produced in a concrete wearing surface is therefore more pronounced than in a concrete tank where the walls and bottom are continuously immersed. Some drying oils, such as linseed oil and tung oil, are often used for the surface treatment of concrete, a usage which seems incompatible with the preceding general statement. These particular oils, known as drying oils, harden when exposed to the air, whereas the majority of other oils becomes rancid and develops acid products. The oils for treatment are applied and allowed to harden by drying and are not exposed to a continuous or intermittent supply of fresh oil.

Surface treatments with sodium silicate or magnesium fluo-silicate and synthetic resin paints are of some assistance, but they are of limited value when applied to the wearing surface of a floor because of the abrasion.

Recommendations regarding the treatment of concrete tanks for storage of various animal and vegetable oils are given in Appendix 17 of the Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, *Proceedings*, American Concrete Institute, Vol. 21, p. 409 (1925).

What heat will concrete withstand without deterioration? (No. 35-28*)

A—In further reference to the effect of high temperature on concrete, attention is called to an investigation conducted at the University of Washington, Seattle, and reported in Bulletin 43, "A Comparison of the Effect of High Temperatures on Concretes of High Alumina and Ordinary Portland Cements" by Alfred L. Miller and Herbert F. Faulkner, in 1927.

The purpose was to compare the effect of high temperatures on the strength of high-alumina cement concrete and of ordinary portland cement concrete under the same conditions. Cylinders were 2 in. in

*See JOURNAL, Amer. Concrete Inst., Feb. 1939; *Proceedings* Vol. 35, p. 292.

diameter and 4 in long, the size limited by the ovens used for treatment. Materials were of ordinary commercial grade. The fineness modulus of sand was 2.71, gravel (maximum size ½ in.) 6.33; fineness modulus combined aggregate 5.12. Seven pounds of water were used with each batch consisting of 11.8 lb. cement, 24.5 lb. dry sand and 50 lb. gravel, with a water-cement ratio of 0.90. Sixty cylinders of each concrete were prepared; cured 24 hours in a moist closet, 9 days immersed in water; later in air at ordinary room temperature. A group of specimens of high-alumina and of ordinary portland cement concrete were placed in an oven which had been brought to the required temperature and removed after one, two and four hours. Temperatures of the treatment were, in degrees Fahrenheit, 200, 300, 500, 800, 1100, 1400, 1700. Heating at 200°F and 300°F was done in a thermostatically controlled electric drying oven. Higher temperatures were obtained in an automatic electric heat treating furnace. The small size of the test specimens doubtless influenced the results, as not indicating exactly the probable results from larger masses. Results are regarded as relative rather than exact. No determinations of free and combined moisture in the cylinders were made. For both concretes the maximum loss in weight was attained at temperatures of 1400°F and above.

For the one-hour group the high-alumina concrete was reduced to the strength of the corresponding group of the ordinary concrete by a temperature of about 550°F. For the two-and four-hour periods at temperatures less than 200°F. the strength of high-alumina was less than the strength of the ordinary cylinders. This relationship prevailed for all temperatures up to approximately 1,000°F. At temperature of 1250°F. for one hour the high-alumina specimens were stronger than the ordinary specimens. Also when heated for two and four hours at temperatures above 1000°F.

For all temperatures and periods of heating high-alumina specimens were changed in color from ordinary light chocolate brown to light buff. Portland cement cylinders though not greatly changed in color showed a tendency to crack and check at a temperature of 800°F. This effect increased with the temperature. At 1700°F. the cylinders were greatly affected and required most careful handling.

The application of heat at all temperatures decreased the strength of both concretes; the relative loss of strength of the high-alumina concrete was greater than that of ordinary concrete at temperatures lower than 1100°F; at temperatures over 1400°F. the percentage of loss of strength of ordinary concretes was slightly greater than that of the high-alumina concrete. For each concrete by far the greater

part of the loss in strength occurred during the first hour at all temperatures. In the series heated for one hour the high-alumina concrete was reduced to less than the strength of ordinary concrete at temperatures between 500°F. and 1200°F. In the two-and four-hour series the strength of the high-alumina specimens was less than the strength of the ordinary portland cement specimens at temperatures between 180°F. and 900°F. Loss in weight of the high-alumina cylinders was greater than the loss in weight of the ordinary cylinders for all temperatures and periods of exposure. No checking or crazing of the high-alumina specimens occurred at any temperature. Portland cement specimens were appreciably checked by temperatures of 800°F. and over. Quenching of the two high-alumina cylinders heated to 1700°F. did not visibly affect them; slight recovery in strength was noted as a result of the treatment. Portland cement specimens at 1700°F. were completely disintegrated by quenching.

Effect of long time mixing (36-50)

Q—Can you refer me to literature on long time mixing of concrete, say from 10 minutes to 3 hours?

A—In the April JOURNAL, p. 421 and in the June issue p. 581, are discussions in reference to time of mixing in connection with safe length of haul for ready mixed concrete. One of the earliest papers on the subject, "The Effect of Time of Mixing on the Strength of Concrete," by Duff Abrams, was published in the A. C. I. *Proceedings* Vol. 14, page 22, 1918. The data in the paper have been extensively used.

From an A. C. I. member we have these comments: "It should not be overlooked that some of the improvement shown in the tests for increased time of mixing was probably due to the time element itself, and not necessarily to the mixing action. In other studies differences in strength have been reported for mixtures allowed to stand for various periods before specimens were molded. These differences were explained by the evaporation of moisture, or by the absorption of water by the aggregates during the standing period, as new changes in water content, prior to the time of molding, are certain to affect the quality of the paste. Changes in water content, subsequent to the time of molding, may not effect a corresponding change in quality of the concrete because water lost by evaporation or absorption may be replaced by air without affecting the solid structure of the mass. This is in contrast with specimens molded after prolonged mixing, where the effective water ratio of the paste is that due to the water remaining after absorption and evaporation."

Bond strength of triangular reinforcing bars? (36-51)

Since reinforced concrete design requirements specify in square inches of cross-sectional area the amount of steel to be used, an important factor is to obtain the highest possible total bond strength to utilize the tensile strength of the steel. The increase in total bond strength from triangular bars in tests made recently at the University of Washington, doubtless is due to the fact that the perimeter of a triangle is greater than that of a circle or a square of the same cross-sectional area. And also when reinforcing bars are cast in a horizontal position there is little adhesive or frictional resistance on the underside of the bar due to one or both of two factors: (1) Adhesion is prevented by a thin film of water which forms on the underside of the bar, and/or (2) A void space beneath the bar, due to the shrinkage or settlement of the cement paste while still in the plastic state. Most of the triangular bars of tests had a smaller percentage of the surface exposed to these factors than do square and round bars.

All factors—length of embedment, time of mixing, w/c ratio, sand and gravel percentages, depth of cover and the bearing area of the testing face—were kept constant. Tests were made using a series of five triangular bars, a round, and a square, all of the same cross-sectional area. The bars were cast both horizontally and vertically for the purpose of comparison.

With the bars oriented horizontally, the total bond strength increased directly as the altitude of the bars increased and the base width decreased. In nearly all cases the strengths were greater than those of the round and square bars. Also, as the total bond strengths increased from added surface contact the unit bond strength decreased. The standard square bar was found to be more satisfactory than the round bar when oriented horizontally.

With vertical orientation, the total bond strength increased in most cases with the increase in altitude, up to the bar which is nearly

Dimensions	Total Bond Hor. Orientation (Lb.)	Total Bond Vert. Orientation (Lb.)
$\frac{1}{2}$" base—2" altitude	11,010	10,585
$\frac{3}{4}$" base—$\frac{4}{3}$" altitude	10,040	13,477
1" base—1" altitude	9,272	14,443
$\frac{5}{4}$" base—$\frac{4}{5}$" altitude	8,835	12,914
$\frac{3}{2}$" base—$\frac{2}{3}$" altitude	7,332	13,781
$\frac{3}{4}$" round	7,125	11,674
$\frac{3}{4}$" square	8,582	11,477

Note: 8 inch embedment, concrete—3800 p.s.i.

equilateral in shape and then the total bond decreased. In this case also the unit bond strength decreased with the increase in altitude. Nearly all of the triangular bars had a higher total bond strength than the round or the square bars.

The total bond strengths of the vertically cast bars were higher, in all but one case, than that of the horizontally cast bars. The difference is probably due to the elimination, due to vertical orientation, of the underside void resulting from horizontal casting.

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Vol. 36 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Grouting with a cement gun

R. C. KEEFER, *Engineering News-Record*, Vol. 122, No. 25, June 22, 1939, pp. 61-62
Reviewed by R. W. BEAL

Details are given for easily converting a cement gun into a pressure grouting machine. Such a machine was used satisfactorily for grouting a stone pier under a railroad bridge after ice damage.

Articulated end spans feature Big Creek concrete arch bridge

Engineering News-Record, Vol. 123, No. 3, July 20, 1939, p. 53 Reviewed by R. W. BEAL

The four-span reinforced concrete arch bridge across Big Creek in California is notable in that the half-arch end spans are hinged both at the abutments and at their bases. This arrangement allows for later adjustments by jacking if the abutments settle as is expected.

Building limes, method for analysis

Zement, Vol. 28, No. 34, Aug. 24, 1939, pp. 521-524. Reviewed by L. T. BROWNMILLER

The method of analysis for limes according to German specifications is given in detail. The method covers the determination of ignition loss, carbon dioxide, moisture, insoluble matter, soluble silica, total R_2O_3, Fe_2O_3, CaO, MgO, combined SO_3, and sulphide sulphur.

Vibration devices

G. HELM. *Zement*, Vol. 28, Nos. 30, 31, Aug. 3, 10, 1939, pp. 467-468, 482-485.
Reviewed by L. T. BROWNMILLER

This article reviews briefly the latest developments in vibrating machines for compacting concrete and similar artificial construction materials. The devices described are only those for application to manufactured articles such as block, pipes, etc.; vibration for concrete roads and structures is not discussed. The sources from which the author derived his data are patents. Drawings and sketches of some of the machines are given.

Possum Kingdom initiates Brazos development

Engineering News-Record, Vol. 122, No. 23, June 8, 1939, pp. 71-75 Reviewed by R. W. BEAL.

On the Brazos River in central Texas is being constructed the first of 13 proposed dams. The structure is novel in that it is the first of the so-called massive-buttress

type of flat-deck buttress dam. It is 189½ ft. high and 2,740 ft. long, with deck spans of 40 ft. Details are given concerning the contraction joints, reinforcement, peculiar foundation features, grouting and methods of placing concrete. All aggregates are conveniently available, with the large sizes being obtained as crushed limestone and limestone crushes dust occasionally being added to the sand.

Review of twenty-fourth general meeting, association of Japanese portland cement engineers

BULLETIN B-24, April, 1938. Reviewed by J. C. PEARSON

This 80-page publication presents abstracts of 19 reports and papers related to the industry in Japan. Most of the papers are in English, some are in German. The nature of the subjects covered is diversified, including three on the constitution of cement, three on kiln and grinding mill operation, and others on various phases of cement and concrete testing, construction methods on pavements and dams, cold weather construction, etc.

Thickened-edge pavements a failure

W. S. DOWNS, *Engineering News-Record*, Vol. 123, No. 5, Aug. 3, 1939, p. 60 Reviewed by R. W. BEAL

In the opinion of the author the present thickened-edge or "balanced design" pavement design, which has been adopted by 41 states, is a failure because the original assumptions involved concerning load distribution and slab behavior are unwarranted. The principal trouble comes from transverse cracks which throw the entire design "out of balance" and which are almost certain to appear in pavement slabs of more than 15 ft. in length. The author points out that such cracking is not so serious if the concrete slab is designed for a uniform thickness.

A low-cost concrete building

Engineering News-Record, Vol. 123, No. 5, Aug. 3, 1939, p. 62 Reviewed by R. W. BEAL

Light porous concrete walls were used in a building at Coulee Center to provide economy of construction, minimum shrinkage, maximum insulating value, walls into which nails can be easily driven and surfaces that will take plaster or stucco directly. The mix consisted of aggregate of almost uniform size ($\frac{3}{16}$ to $\frac{3}{4}$ in.), three sacks of cement per cu. yd. and enough water to produce a dry mix which would "stand alone." It is hoped by the designer of the mix, O. G. Patch of the Bureau of Reclamation, that this type of construction may be developed for use where the standard concrete construction would be too expensive.

Observation at Boulder

Engineering News-Record, Vol. 123, No. 1, July 6, 1939, p. 63 Reviewed by R. W. BEAL

At the request of *Engineering News-Record* the Bureau of Reclamation has prepared a statement indicating that the periodic research observations on more than 1,300 installations of measuring equipment on Boulder Dam show no deterioration of concrete, no cracking of structural significance, no differential movements of elemental blocks, and no disturbances due to the recent minor earthquake shocks which have been attributed to the filling of Mead Lake. The measurements to date have checked very satisfactorily with design analyses and the operation and service record of the dam has been entirely adequate.

Architectural concrete surface for Los Angeles Terminal

C. W. PIERCE, *Engineering News-Record*, Vol. 123, No. 1, July 6, 1939, p. 64
Reviewed by R. W. BEAL

A concrete surface of special interest because of its attractive appearance, glare-reducing properties and freedom from crazing has been produced on the exterior and interior walls of the new Los Angeles Union Passenger Terminal by lining the forms with Celotex fiber board of the roughest commercial texture. The $\frac{1}{16}$ in. thick Celotex boards were carefully placed so as not to show joints and nail head marks. A thin brush-coat of boiled linseed oil was allowed to stand on the Celotex surface for several days before placing the concrete. At a maximum of five days after the concrete was placed the forms were removed and the Celotex lining loosened and shifted slightly, after which it was kept wet to provide adequate curing conditions.

Protective coating for concrete placed in forms before pouring

Engineering News-Record, Vol. 122, No. 23, June 8, 1939, pp. 85-86 Reviewed by R. W. BEAL

Protective coatings on the under side of concrete wharves in San Francisco Bay are being placed before the concrete is poured. Immediately after the forms are completed and before the reinforcing is put in place, a layer of heavy building paper is layed in the forms and this is covered with $\frac{3}{8}$ to $\frac{1}{2}$ in. of hot asphalt upon which is sprinkled clean pea gravel. After the asphalt has cooled and any loose pebbles are swept off, a good surface is left for bonding to the concrete. The reinforcing bars are next put in place and the concrete poured. After 15 years of use the method still appears very satisfactory and is very inexpensive because the presence of the protective coating materials on the forms allows for the use of knotty, cheap lumber for forms.

Crazing of cast stone

ANON. *Concrete Building and Concrete Products*, Vol. 14, No. 8, p. 148, Aug. 1939.
Reviewed by J. C. PEARSON

Summarizes briefly the investigation of the Building Research Station (England) on this subject, as described in the 1938 annual report. Tests over a period of years have shown that if a block of concrete is long exposed to the atmosphere, the chemical composition of the surface layer is changed by the action of acid gases, such as CO_2 and SO_2. The latter has little effect on shrinkage, but carbonation is accompanied by a shrinkage which is about $\frac{2}{3}$ of the initial wet-to-dry shrinkage of the concrete. This source of crazing can be offset to some extent by artificially carbonating the stone while it is still green, especially if the stone is somewhat porous. In the case of dense stone the advantage of the treatment has not been proved on a commercial scale, and in any case would not necessarily overcome the liability to crazing.

A new vibratory machine for determining the compactibility of aggregates

GARBOTZ-SCHADE, *Die Betonstrasse*, V. 14, No. 8, pp. 174-175, Aug. 1939. Reviewed by F. L. EHASZ

An account of the vibratory machine reported on by V. T. Pauls and J. F. Goode in the May 1939 issue of *Public Roads* is given in this German article. This machine is believed to yield more consistent results and higher densities, at the same time being useful for all gradations of aggregates employed in base and surface courses. Recapitulating in the words of the original paper, the vibratory test comes into play in the following problem: 1) establishment of a definite optimum degree of compaction toward which field compaction may be aimed; 2) determination of the best

combination of two or more available aggregates for base-course or surface construction; 3) investigation of the capacity for bituminous materials of certain aggregates to insure against over-bituminization; 4) modification of aggregate blends to permit the use of sufficient bituminous material for workability and surface sealing without overfilling the void spaces and destroying stability.

Raising O'Shaughnessy dam

Engineering News-Record, Vol. 122, No. 21, May 25, 1939, pp. 57-59 Reviewed by R. W. BEAL

In the initial structure of the O'Shaughnessy Dam in California, completed in 1923, were incorporated plans for later raising the crest of the dam 85 ft. The major problem was that of getting the proper bond between the downstream face of the initial structure, arch-gravity type, and the new section so that the final composite structure would act as a homogeneous unit. This problem is believed to have been solved satisfactorily by supporting the new section along the face of the initial structure with ribs until the new concrete was essentially completely cured and set, circulation of cooled water through embedded pipes having been used to carry away the heat. Square steel reinforcing bars, spaced 2½ ft. horizontally and 5 ft. vertically and each with one end embedded 5 ft. into the old concrete and the other end 5 ft. into the new concrete, were used to provide added bond. The intervening spaces between the new and old sections were then filled with concrete to complete the job.

Mineralizers in cement

E. P. FLINT, *Rock Products*, Vol. 42, No. 10, Oct. 1939, pp. 40-42 and 52. Reviewed by ROY N. YOUNG

This paper is a progress report on studies by the National Bureau of Standards on mineralizers in the manufacture of cement. Results of tests indicate the relative mineralizing activities of magnesium fluosilicate, calcium fluoride, calcium phosphate, boric oxide and cryoline, when used in cement raw mixtures containing small proportions of R_2O_3 (from 0 per cent to 3 per cent $Al_2O_3 + Fe_2O_3$). The effectiveness of a mineralizer was judged by the observed contents of $2CaO \cdot SiO_2$, $\beta + \gamma 2CaO \cdot SiO_2$, and free CaO in the resulting clinkers after one or more heat treatment. A treatment consisted of heating the raw mixture for one hour at 1350°, 1400° or 1450°C.

The effects of boric oxide and the phosphate were not beneficial but probably detrimental. No conclusions were drawn with respect to the cryolite. Calcium fluoride was effective in promoting reactions but increased the tendency of $\beta 2CaO \cdot SiO_2$ to invert to the γ-form.

Magnesium fluosilicate was the most promising material used, both with respect to its effectiveness and commercial availability. It is suggested that its use may be applied in the manufacture of normal cement mixtures to accelerate reactions of lime and silica, thus lowering the amount of fuel required, also in the manufacture of white portland cement and sulfate resisting cement where lower contents of Fe_2O_3 and Al_2O_3 are required.

Consolidating concrete by vibration

ANON. *Concrete Building and Concrete Products*, Vol. 14, No. 8, p. 145, Aug. 1939.
Reviewed by J. C. PEARSON

The findings of the Building Research Station (England) in its investigation of vibration placing methods as contained in its 1938 report, published in June of this year are summarized. The conclusions, somewhat condensed, are as follows:

1. To be effective, vibration must have an acceleration above a certain critical value, depending on the mix. For all practical purposes a value of 4 g is adequate.

2. Frequency appears of small importance compared with acceleration. High frequencies give a somewhat better surface appearance than low frequencies at the same acceleration.

3. Much harsher and dryer mixes can be placed by vibration than by hand.

4. Grading of aggregate is of small importance for 1:6 mixes and richer. For high workability the sand content can be lower than for hand compacted mixes.

5. Seven-day compressive strength of 3000 p.s.i. can be obtained with 1:9 mixes by careful grading and vibration.

6. There appears to be no important effect on the strength of concrete if subjected to short period vibration subsequent to the original placement by vibration.

7. The improved properties of vibrated concrete—higher strength and density, lower creep and shrinkage, greater bond strength—are all due to the lower water content that can be used with this method of placement.

Ruminations on pavement joints

H. J. GILKEY, *Engineering News-Record*, Vol. 123, No. 3, July 20, 1939, p. 77

also

Supplementary Ruminations on pavement joints

H. J. GILKEY, *Engineering News-Record*, Vol. 123, No. 7, Aug. 17, 1939, p. 42 Reviewed by R. W. BEAL

Proposes a light H-beam type of load transferring transverse pavement joint with a compressible layer of cork or Celotex material adjacent to the web on one side. Beam to be pre-curved to exact surface-of-pavement cross-section and rigidly fixed in position to serve as a finishing guide during casting. Advantages claimed are that it makes full depth of pavement available for load transfer (instead of half-depth as when dowels are used), thereby reducing stress in slab and producing greater stiffness with decreased vertical amplitude of motion under traffic. Joint is water-tight, insuring greater subgrade support. Joint cannot fill with dirt and joint filler cannot extrude. Slab edges cannot curl or get out of alignment, thus insuring smooth riding and freedom from impact stress. Fatigue stresses should be much less severe than for ordinary joints with their inadequate edge support, high impact and relatively large amplitudes of vertical motion. The joint could be used with thickened-edge pavement construction almost as easily as with pavements of uniform depth.

Measurement of air voids in fresh concrete

Technical News Bulletin, National Bureau of Standards, No. 267, July, 1939.
HIGHWAY RESEARCH ABSTRACTS

It is generally recognized that if concrete is improperly placed, the resulting hardened concrete may contain large quantities of air, which will have definite harmful results. It is not so well recognized, however, that air in appreciable quantity is present even in well-placed workable concrete, and that air may be regarded as much a constituent of average concrete as cement, water and aggregate. Consequently, a knowledge of the amount of air present in a given material, the manner in which it is distributed, and its influence on any particular property is a matter of importance.

The quantity of air voids may be computed from specific gravities and weights of the materials in the concrete, or may be measured directly through displacement of the air with water.

A modification of this latter method, recently developed by George L. Pigman at the National Bureau of Standards, is believed to be more accurate than previous procedures, besides being more simple and rapid. In this new procedure the concrete

is placed in an airtight mold and violently agitated with an excess of water while under an absolute pressure of 15 mm. of mercury. The volume of air removed is determined by observing the decrease in the total volume of the concrete after this treatment. It has been found that this treatment removes from 20 to 30 per cent more air from the concrete than can be removed by methods not utilizing the reduced pressure.

Tests with various concretes indicate that the air content is increased with dryness and with increase in fine material in cement, sand, or admixtures.

Construction plant planning for large dams

R. T. COLBURN, *Civil Engineering*, Vol. 9, No. 8, August 1939, pp. 459-462 Reviewed by J. R. SHANK

The author who is construction plant engineer for the Tennessee Valley Authority indicates in this article the advantages of a centralized control of plant for a large complex construction project.

A properly designed construction plant ready to go is a job one-third completed.

A Construction Plant Division of the Construction Department was created to design or purchase, and control all plant and equipment. Studies were made on equipment costs per net operating hour and records were kept on upkeep costs. When the rate of upkeep cost approached that of the net operating cost the equipment was removed by selling or scrapping. Studies and forecasts were made so as to use the equipment completely in one or more jobs and to keep the peak requirements of equipment for the whole project to a minimum. Even dwelling houses were moved from job to job in some cases. Travelling cableways were moved from job to job. The cables were systematically greased and rotated one-eighth turn monthly in order to keep the wear evenly distributed. Each job was held responsible for efficient operation and maintenance of the equipment assigned and charged to it.

An interesting feature of the equipment at the Wheeler Dam was a mixing plant on a barge with small storage for aggregate and cement. A whirely crane on the barge served to supply these storage bins from supply barges. A 6-yd. bucket with pneumatic opening mechanism provided facilities for controlling the cable bounce and added to the safety of its operation.

The hydraulic properties of blast furnace slags

G. MUSZGNUG. *Zement*, Vol. 28, Nos. 16, 17, 18, 19, April 20, 27, May 4, 11, pp. 248-253, 261-268, 279-283, 293-298. Reviewed by L. T. BROWNMILLER

This paper is poorly titled for its value lies in what may be a method for testing slags but not in a study for determining the hydraulic properties of slags.

The author's main conclusion is that the hardening rate of high basic slags is more dependent on the heating conditions than on the chemical composition. Yet he has spent a considerable time analysing slags chemically and relating composition to strength with hardly any mention of heat treatment except a casual estimate of glass content. The data which he gives are comprehensive analytical and strength figures. The strengths were obtained by two methods of testing which depart apparently from the usual method of diluting the slag with cement and subjecting the mixture to cement tests. It is beyond this reviewer's province to determine if there is a need for a test on practically neat slag.

By the methods of testing outlined, about 1 per cent of a catalyser is added to the slag. The catalyser may be lime, gypsum, a combination of the two, or portland cement. By the first method the slag is mixed with 10 per cent of water and compressed into cylinders under high pressures which are subsequently tested for

strength. In the second method the slag is mixed with 30 per cent of water and molded into prisms for strength and expansion.

The specimens are steamed for four hours and then cured in water or air or a combination of water and air curing until the time of test. The strengths of a very large number of specimens were obtained and are given in graphs and tables.

In addition, the heat evolution of slags was determined by the method of Grun by measuring the temperature rise in a vacuum flask. The amount of heat evolved during the first 36 hours of hardening is small as compared to that of portland cement.

Special cements

C. R. Platzmann, *Cement & Lime Manufacture*, Vol. 12, No. 8, p. 163, Aug. 1939.
Reviewed by J. C. Pearson

Presents a brief review of the properties of the numerous cements now on the European market, and at the same time indicates that the number of special cements exceeds all reasonable limits. The classification is as follows:

1. Portland Cement: Standard portland; High-early strength, Ore cement; White cement; Low heat cement.

2. Slag-containing Cement: Iron portland; H. E. S. iron portland; Blast furnace cement; H. E. S. blast furnace cement.

3. High Alumina Cement.

4. Mixed Cements: Pozzalanic; Trass (2 types); Gaize; Moler (low heat).

5. Special Cements: Quick-setting; Masonry; Slag (rich in sulphates); Water-repellent.

The author comments as follows: "It is seen from this list that, starting from the original portland cement, some twenty types of cement have been developed. It is not easy to understand why a problem, originally simple, has been complicated in this way, and it may be pointed out that normal portland cement still accounts for approximately 80 per cent of the worlds production. It would seem that some of the newer cements are superfluous. It is not certain that the so-called higher-early strength cement is a necessity, because the one-sided over-evaluation and development of compressive strength is of no advantage so long as the tensile strength is not increased in the same degree. Masonry cements can also be considered superfluous because lime-cement mortar has for decades achieved the same purpose. With regard to quick-setting cement, it may be mentioned that quick-setting compounds have been available for many years, and these are sufficient. Slag cement rich in sulphates cannot be considered as indispensable, because ore cement, aluminous cement, and the different types of pozzolanic cements seem to fulfil the same purpose."

Another instance of unreliability of the sodium sulfate soundness test

A. T. Goldbeck, Engineering Director, N. C. S. A., *Crushed Stone Journal* for July-August, 1939 Vol. XIV No. 4.
Reviewed by J. E. Gray

The sodium sulfate soundness test as applied to coarse aggregate is intended to be an accelerated test for determining the resistance of aggregate to the destructive effects of freezing and thawing. Stone for use in trickling filters is commonly required to withstand twenty cycles of the sodium sulfate test with a loss of less than ten per cent. A sample of stone was obtained from a dismantled trickling filter which had been in service for twenty-five years and had given satisfactory performance. This sample when subjected to twenty cycles of the sodium sulfate soundness test showed a loss of forty-nine and nine tenths (49.9) per cent.

Objections may be raised that had the stone been tested before exposure the results might be different. When another laboratory tested the exposed stone and also a sample supplied by the producer representing the original material, the percentages of loss of these two samples were practically the same.

Reference is made to a paper* by Messrs. Cantrill and Campbell in which it is definitely proven that chert gravels from the Tennessee and Cumberland Rivers have given poor results when used as coarse aggregate in concrete roads, yet they appear sound when tested by the sodium sulfate soundness test. On the other hand, the limestone on an average showed a loss of 12.1 per cent in five cycles and could be considered unsound, yet it has given excellent service in concrete roads. The conclusion is drawn that enough lack of parallelism between test results and service behavior is on record to warrant a move toward discarding the sodium sulfate soundness test altogether.

A quick method for estimating the absorption of fine aggregate

HORACE A. PRATT, Bulletin No. 35, June, 1939. Maine Technology Experiment Station.
HIGHWAY RESEARCH ABSTRACTS

Certain routine tests of sands may be greatly expedited if the absorption can be determined accurately enough in considerably less time than that required by the A. S. T. M. Method C-128-36 T.

As a result of investigations of 60 Maine sands the author has arrived at the conclusion that a reasonably satisfactory prediction equation based upon the joint influence of various physical characteristics of the sands upon the absorption can be derived, the use of which will obviate the necessity of making the regular absorption tests on sands.

Although an equation derived for Maine sands would not necessarily apply to the sands of other localities, still the same method would apply and a similar equation could be prepared from a like study of the sands of any region.

In particular, this study indicates the following facts:

1. The amount of water a sand will absorb may be estimated by any of the three following equations: (I) Estimated per cent absorption = 1.45 − 0.0147 (per cent granitic) + 0.3242 (fineness modulus); (II) Estimated per cent absorption = 3.07 −0.143 (per cent granitic) + 0.0005 (surface area); (III) Estimated per cent absorption = 1.72 − 0.0148 (per cent granitic) + 0.0105 (per cent retained on 28-mesh sieve).

2. Practically no differences exist between the respective multiple correlation coefficients, the standard errors of estimate, or the percentages of variation controlled. This indicates that equally satisfactory results may be obtained with each of these formulas.

3. The small differences which are present slightly favor Equation II for prediction purposes.

4. The chances are fifty-fifty that the per cent of water a sand will absorb will be within 0.34 of the estimated value predicted by Equation II.

Portland cements which are not seriously affected by large additions of water

ANONYMOUS *Cement & Lime Manufacture*, Vol. X11, No. 6, June 1939, pp. 115-129
Reviewed by J. C. PEARSON

An abstract of a report by Prof. Serban Solacolu of the Technical College of Bucharest on an extensive investigation of the properties of laboratory burned

*Selection of Aggregates for Concrete Pavements Based on Service Records by Curtis Cantrill and Louis Campbell. Presented at the 42nd annual meeting of the American Society for Testing Materials, June 26-30, 1939.

cements discusses at length the effect on strengths of changes in W/C from 0.3 to 0.5 for different cement compositions. Since the endeavor was made to eliminate in so far as possible the effects of degree of burning, treatment of clinker, fineness, uncombined lime, etc., the author justifies a method of charting results in which silica modulus, *Ms*, (ratio of silica to iron oxide plus alumina) is plotted against alumina modulus, *Ma*, (ratio of alumina to iron oxide). Thus each cement is represented by a single point on such a diagram, in which the region at the left contains the high iron cements, that at the right the high alumina portlands, and between these the normal portlands. In this central zone, the high silica portlands are toward the top and the low silica portlands toward the bottom. On such a diagram lines may be drawn through those points which represent cements having equal setting times, equal normal consistencies, equal tensile strengths, etc.

The reduction in strength for any cement from a water-ratio of 0.3 to a water-ratio of 0.5 may be expressed algebraically as

$$\Delta = \frac{S_{0.3} - S_{0.5}}{S_{0.3}} \times 100$$

and this function may also be plotted on the type of diagram described. If a series of lines is then drawn through points representing cements having the same value of Δ, the sensitivity of the different compositions to the effect of excess water is seen at a glance. The conclusions are as follows:

1. Portland cements with *Ms* values from 1.8 to 2.5, and *Ma* values from 1.5 to 2.4 are more insensitive than the high iron cements. Strength reductions for the higher water ratio vary for this group from 5 to 50 per cent.

2. High iron cements with a value of *Ma* less than 1.5 are sensitive to excess water, the strength reductions varying from 60 to 80 per cent.

3. Alumina-portlands (*Ma* greater than 2.5) are more sensitive to excess water than normal portlands.

4. Portlands which are least sensitive to water additions are those which have a silica modulus of 2.00 to 2.10, and an alumina modulus of 2.00. The strength reduction for the higher water ratio in this class varies from 5 to 15 per cent.

5. Calcium and aluminum chlorides reduce the sensitiveness of cement to excess water. In excess of 3.5 per cent, calcium chloride practically causes the sensitiveness of excess water to disappear.

6. A connection is noted between special cements insensitive to water and the oil well cements used in America and Rumania.

Aggregate selection and concrete mix design

CHARLES E. WUERPEL, *Civil Engineering*, Vol. 9, No. 9, September 1939, pp. 539-542.
Reviewed by J. R. SHANK

The characteristics and economy of a concrete structure are profoundly affected by the judicious selection and use of aggregate. Remoldability tests show that whereas a smooth round gravel may work best with a 32 per cent ratio of sand to total coarse aggregates, angular crushed granite of the same size grading may require 47 per cent. Further investigations of these two materials developed a mix of 1:2.45:5.75, requiring 4.55 sacks of cement per cu. yd. for the former and 1:2.60:2.90 with 6.15 bags of cement for the latter.

Grading must be properly worked out and consistently maintained even though it may be necessary to train producers and provide proper handling and storage facilities. The limits of permissible variation of grading of fine aggregates should be

narrower than they usually are and in no case should the sand contain less than 10 per cent passing the No. 50 sieve or less than one per cent passing the No. 100 sieve.

The segregation of water is of greater importance than the segregation of coarse aggregates and is caused by insufficient surface area being presented by the cement, sand, and coarse aggregate to hold it by surface tension. This form of segregation is also called "water gain," "water rise," "bleeding," or "sand streaking." Segregated water forms under aggregate particles causing weakness and permeability and under reinforcing steel causing destruction or absence of bond, and danger of corrosion.

The structural soundness of aggregate must not be overlooked. It is rather simple to detect unsoundness in ledge rock but glacial and alluvial gravels present more difficult problems. The magnesium sulfate accelerated soundness test is recommended.

Aggregates affect the structural qualities of hardened concrete in a number of ways: to vary the modulus of elasticity from 2,000,000 to 6,000,000 p.s.i., the coefficient of temperature expansion from 0.000004 to 0.000006 in. per in. per degree F., the flexural strength, specific heat, thermal conductivity, and coefficient of heat diffusion and other volume change characteristics such as plastic flow, shrinkage of curing, and humidity variations.

The trial method for determining the mix with the aid of remoulding apparatus, in view of the particular conditions of transportation, placing and consolidation, and of the shape, size, and utility of the monolith is preferred.

Internal vibration is one of the major advances in the concrete industry, but it must be used intelligently. Whether or not vibrators are to be used must be considered when making up the mix. A drier mix should be used with vibrators to avoid danger of water segregation. It is questionable whether vibrators should be considered when placing concrete around intricate reinforcing steel where it is imperative that an easy flowing mix be used. During the period of vibration and in the vicinity of the vibrator full hydrostatic pressure from 150 lbs. per cu. ft. should be expected and the forms should be made to withstand such pressures. After vibration ceases the mass stiffens very quickly causing rapid release of lateral pressures.

The hydration of the cement produces a volume which is less than that of the original unhydrated cement plus the surrounding water. This causes a drawing of water inwardly into the concrete so that if water is not supplied at the surface a portion of the concrete near the surface will be deficient in water for later hydration. It is therefore necessary to supply this water from external sources and maintain the supply constantly for as long as conditions will permit. There is no substitute that can take the full place of an excess of water on the surface of the concrete.

Colored concrete

Road Abstracts, Vol. V, No. 10, Jan., 1939. From HIGHWAY RESEARCH ABSTRACTS

Colored concrete is now extensively used for marking traffic lines, in marginal strips or parking areas, and in ordinary carriageway surfaces to diminish glare. The pigment may be incorporated in the cement at the time of manufacture or may be added to the concrete in the mixer or applied to the surface of ordinary concrete after the latter has been placed. The pigment employed should be very finely ground; it should not deteriorate when mixed with cement or as a result of weathering.

The most suitable materials are certain metallic oxides, notably those of iron, chromium and manganese and a few other inorganic materials such as ultramarine, and lamp black. Iron oxide pigments, which may be either natural or artificial, range in shade from yellow to black; greens are produced by admixtures of chromic

oxide or hydroxide, and blues with ultramarine. Black pigments include certain forms of ferric oxide, manganese dioxide, and lamp black. The latter should be free from sulphur compounds, oil and traces of sugar. Naturally occurring iron oxide pigments are used but corresponding artificial products are considered preferable, since they are free from traces of soil, etc., which weaken the concrete; the composition of artificial pigments, moreover, is more constant than that of natural products. Pigments should be tested for fineness, coloring power, permanence on exposure to light and when mixed with cement, and freedom from organic coloring matter and sulphur compounds.

Varying opinions have been expressed regarding the influence of pigments on the strength of concrete. The influence of varying proportions of pigments on the strength of cement and concrete of stated composition has recently been investigated in Germany in a number of laboratories. Tests carried out in 1936 included the determination of the beam-tensile and compressive strengths of the specimens at 7 days, 28 days, and 3 months; observations of weight and dimensions during air storage at 20°C. in an atmosphere of relative humidity, 60 per cent; alternate exposure to water, frost, ordinary air, and heated air, of composite beams, the lower part consisting of plain and the upper part of colored concrete; outdoor exposure to test permanence of color; and tests on the actual pigments. The colors used were two types of iron oxide (Black and brown); a coarse and a fine lamp black and a special black pigment. It was found that all specimens maintained reasonably constant dimensions. The setting time was not influenced by the iron oxide pigments; it was slightly slower with the coarser lamp black and was considerably so with the fine lamp black and the special black pigment. The tensile strength of standard cements was not perceptibly influenced by 5 per cent of the iron oxide colors or of the special pigment, and only slightly by 8 per cent. Five per cent of fine lamp black increased the strength by 10 per cent and 5 per cent of the coarser lamp black by 2.5 per cent; 8 per cent of lamp black caused deterioriation in the concrete. The beam and compressive strengths were only slightly influenced, the finer lamp black causing a slight increase. The resistance to alternating stress was not influenced by any of the pigments, and the composite specimens subjected to alternate exposure for three months were not injuriously affected. None of the pigments showed fading at the end of three months.

In 1937 similar tests were applied to a considerable range of black and brown pigments, including a natural ochre brown and three carbon blacks derived from coke. Dimensions again remained constant. The setting time was slightly accelerated by some pigments and somewhat retarded by others, especially by lamp black. Any changes in the tensile and compressive strengths of cement were within the limits of experimental error; it is noted that this result includes fine lamp black, which considerably increased the tensile strength in the earlier series of experiments.

The use of colored cement is recommended, as it is difficult to secure uniform mixing on the site, and imperfect mixing results in streakiness. Special provision must be made for transport and storage, as colored cement must not be allowed to become mixed with standard cement. If it is necessary to add the pigment on the site, the cement and the coloring matter should be premixed in a special concrete mixer, or the pigments should be mixed with dry fine quartz sand. The color obtained can be enhanced by the use of dark aggregates, the grain of which should preferably be coarse in order to diminish reflection. The surface of the fresh concrete should be broomed and belted immediately after construction, and must be kept wet for several weeks. The color of concrete surfaces is liable to become obscured by efflorescence, which may, however, be reduced by using cement which is thoroughly burnt and well matured, and by attention to cleaness of the sand and purity of the mixing water.

An approach to the design of concrete mixes

CHARLES E. WUERPEL, *Civil Engineering*, Vol. 9, No. 8, August 1939, pp 463-466
Reviewed by J. R. SHANK

Hardened concrete is a coherent conglomeration of materials blended and molded in such a manner as to meet specific structural requirements. Good concrete is that in which the blending of the component materials effectively meets the particular structural requirements of the member it forms.

After becoming thoroughly familiar with the nature of the project, the first step is the choice of the water-cement ratio which should be the highest value commensurate with the demand for strength, density and durability. The second step is the selection of the cement.

The rates of heat generation of the four constituent compounds of portland cement are in the order, beginning with the most rapid; C_3A, C_3S, C_2S and C_4AF. The rates of strength gain are in the same order excepting that the C_3S and the C_3A should be interchanged. Cements high in C_2S shrink more than those high in C_3S. Cements high in C_3A show considerable expansion and contraction during hydration. C_3A is also the compound most affected by contact with aggressive water. The reduction of C_3A and C_3S is most essential when conditions demand low heat generation or high resistance to chemical attack. It must, however, be noted that reduction of these two produces slower strength development. An increase of C_4AF which contributes little to the desirable qualities follows a decrease of C_3A for reasons connected with the manufacture. The conditions which demand special cements of either the low heat slow strengthening, or high heat rapid strengthening kinds are so special that the ordinary cement is the thing to use for 75 per cent of all construction.

The Federal Government has recently issued four portland cement specifications which cover the requirements for special cements.

SS-C-191a is a cement suitable for normal building construction and for non-massive structures not exposed to appreciable climatic or chemical attack.

SS-C-206 is the "moderate heat of hardening" cement. The requirement for fineness is such as to make it excellent for workability and plasticity. It is high in resistance to climatic and chemical attack. (This cement is very similar to the 15-C cement used by New York State and is the type generally used by the Corps of Engineers in flood control work in the Northeastern United States.)

SS-C-211 is known as a sulphate-resisting cement and is designed for use in cases where attack by alkali or sea-water is expected to be particularly severe.

SS-C-201 is known as high early strength cement and is designed for use in structures which must be placed in service in the shortest period of time and where exposure to climatic and chemical attack will not be severe.

CHEMICAL REQUIREMENTS AND SURFACE AREA REQUIREMENTS—
FEDERAL SPECIFICATIONS FOR PORTLAND CEMENT

Specification	SiO_2	Al_2O_3	$\frac{Al_2O_3}{Fe_2O_3}$	C_3A	Fe_2O_3	MgO	SO_3	Surface Area
SS-C-191a	..	−7.5%	....	−15.0%	−6.0%	−5.0%	−2.00	+1,500
SS-C-206	+21%	−6.0%	0.7 to 2.0	− 8.0%	−6.0%	−5.0%	−2.00	+1,800
SS-C-211	+24%	−4.0%	0.7 to 2.0	− 5.0%	−4.0%	−4.0%	−1.75	+1,800
SS-C-201	..	−7.5%	....	−15.0%	−6.0%	−5.0%	−2.50	+1,900

NOTE: A minus sign indicates "not more than;" a plus sign, "at least." Surface area in square centimeters per gram.

Puzzolanic materials are added to or partly replace portland cements on the dual premise that they will react with the calcium hydroxide liberated from hydrated C_3S to form calcium silicate, and that they will lessen the rate and amount of temperature rise.

Natural cements are occasionally used as a replacement for a portion of the portland cement, either to minimize temperature rise or to increase resistance to deteriorative forces.

Alumina cements such as "Lumnite" are the most rapid strength gainers and are practically impervious to attack by aggressive water which are the enemies of portland cement. They also evolve more heat and at the most rapid rate, which if not dissipated may destroy cementation. They cannot be blended with portland cement. They cost about 3½ times the cost of portland cement.

If the quality of concrete could be reduced to proportional influences of the factors which enter into its manufacture, it might be possible to state the case as follows: Workmanship, 70 per cent; aggregate, 15 per cent; and cement, 15 per cent.

Proceedings of the symposium on the chemistry of cements, Stockholm, 1938

Published by Ingeniorsvetenskapsakademien, Stockholm (32 Swedish Crowns, approximately $8.00 which price includes postage) Reviewed by R. H. Bogue

This volume of 578 pages includes the complete transactions of the Symposium on The Chemistry of Cement, held in Stockholm on July 6, 7, and 8, 1938, under the joint auspices of the Royal Swedish Institute for Engineering Research and the Swedish Cement Association. All papers and discussions are printed in full, together with a review of the Symposium by the Secretary, a list of delegates and guests, a resolution to the memory of the late Henry LeChatelier, report of a meeting of the International Subcommittee on Special Cements for Large Dams, a Survey of Contents in English, French and German, and an Index.

At the Symposium, delegates representing 11 countries were present. Twelve papers were presented by invited speakers and both prepared and extemporaneous discussions were delivered. Very brief reference to the subject matter of these papers are as follows:

T. Svedberg, University of Upsala, Sweden: "The Study of Giant Molecules by Means of Ultracentrifugal Sedimentation, Diffusion and Electrophoresis." Professor Svedberg's paper had to do chiefly with a description of the ultracentrifuges developed in his laboratory and was largely along the lines given in his paper at the dedication of the Chemistry Building of the University of Delaware and published in the *Ind. Eng. Chem.*, Analytical Edition, March 15, 1938 (Vol. 10) page 113. One of these machines is designed to operate at 500 to 15,000 and the other at 15,000 to 750,000 times gravity. Their application has been almost entirely with organic materials, proteins, biological preparations, etc. But it is not improbable that knowledge of the pattern of the molecules and of the active groups in a unit molecule of the cement compounds may give information not heretofore available on the course of reactions in cements.

J. A. Hedvall, Technical University of Goteborg, Sweden: "Reactions between Substances in Solid State with Special Regard to Systems Containing Silica." Professor Hedvall's paper was a technical discussion of the mechanism of reactions in the solid state and the factors determining the reactivity. He pointed out that, in reactions between crystals, the primary compound through which diffusion takes place between the reacting phases need not be the final compound that is formed. Reaction sets in when the lattice vibration energy is sufficently great and for this reason the reaction temperature is a constant for many reactions of the same reactant. The lattice energy can be profoundly affected by polymorphic transformations and by impurities. The formation at low temperatures of an incipient amorphous reac-

tion layer in a mixture of crystals leads to a maximum in their catalytic and chemical properties. Reactions occurring in the formation of portland cement were given as a case in point. When CaO and SiO_2 are heated in the solid state, the orthosilicate is formed rather than the metasilicate, due to a higher resulting heat content, a smaller required activation energy, and easier crystallization of the orthosilicate. Nucleus formation and velocity of growth are better, and the structural similarity between raw materials and reaction products is better. In orthosilicates each Si atom is surrounded by four O atoms to form a tetrahedron, while with metasilicates long open chains are formed.

R. H. Bogue, Portland Cement Assn., Fellowship, National Bureau of Standards, Washington, D. C.: "Constitution of Portland Cement Clinker." Dr. Bogue's paper was concerned mainly with a presentation of the more recent developments in the art of clinker examination, in the findings by phase equilibria studies on the alkalies, and in the character and effects of glass. The microscopic methods of examination of polished sections as developed by Insley and Ward were given and the character of the alkali and glass phases as revealed by such study was discussed. The nature of the glassy phase as found by Brownmiller, its method of approximation by the procedure of Lerch and the significance of the glass on the composition of the crystalline phases were given together with equations by Dahl for calculations of the phases under certain assumptions. Recent unpublished results obtained by Taylor on the possible composition of the alkali phases were presented.

W. Bussem, Kaiser Wilhelm Institut fur Silikatforschung, Berlin, Germany: "X-Rays and Cement Chemistry." Dr. Bussem presented his paper in two parts, first the results that have been achieved in the study of cement chemistry by the use of X-rays and second, since a further penetration into unexplored territory is possible principally by the use of improved methods, a discussion of the methodical methods of X-ray investigation. The author presented his views of the coordination principle as a basic and orderly principle of crystal chemistry.

Suggestions were offered which might lead to an improvement in the accuracy and sensitiveness of X-ray analysis.

G. E. Bessey, Building Research Station, Garston, Herts, England: "Calcium Aluminate and Silicate Hydrates". Mr. Bessey's paper was for the most part a correlation of many previously published investigations by numerous authors on the two separate systems $CaO\text{-}Al_2O_3\text{-}H_2O$ and $CaO\text{-}SiO_2\text{-}H_2O$. He described first the naturally and artificially prepared compounds in these systems, their methods of preparation and properties. Following this he gave an analysis of the solubility and phase equilibria data that have been presented and an attempt at an explanation and classification of the results.

T. Thorvaldson, University of Saskatchewan, Saskatoon, Canada, "Portland Cement and Hydrothermal Reactions". Professor Thorvaldson considered the reactions occurring between water at 100°C. or higher and the silica, alumina and ferric compounds of portland cement. The chief difference between the action of water and saturated steam on the calcium silicates at high temperatures and the hydration of these at ordinary temperatures, apart from the time required for the reactions, is believed by the author to be the formation of crystalline as against gelatinous calcium hydrosilicates. In addition the hydrothermal treatment favors direct hydration of the silicates to form hydrated products with a lime-silica ratio not lower than 2, or possibly somewhat higher for $2CaO.SiO_2$ in presence of $Ca(OH)_2$ and even approaching 3 for $3CaO.SiO_2$ in contact with the vapor phase only at high temperatures. On the other hand, hydration at ordinary temperatures, especially

in the presence of an excess of water, tends to give lower ratios of combined lime to combined silica, although in cement pastes the equilibrium ratios for dicalcium silicate may almost reach 2 and even higher for tricalcium silicate. The favorable effect of steam curing on strength and sulfate resistance of mortars is considered to be due to reaction between the free lime liberated by hydrolysis and the siliceous aggregate, the direct formation of crystalline calcium silicate hydrates and the conversion of the aluminates to stable crystalline $3CaO \cdot Al_2O_3 \cdot 6H_2O$.

P. Schlapfer, Eidgenossische Materialprufungsanstalt an der E. T. H., Zurich, Switzerland: "Reactions of Portland Cement with Water." Dr. Schlapfer discussed first the action of water at ordinary temperatures on the cement compounds and concluded with a consideration of the summation of effects of water on portland cement. The silicates of the cement produce $Ca(OH)_2$ and a gelatinous hydrate containing $3CaO \cdot 2SiO_2$. Gypsum reacts with the aluminates to form calcium sulfoaluminate, finely dispersed along the outer surfaces of the aluminate. The $4CaO \cdot Al_2O_3 \cdot Fe_2O_3$ is decomposed to finely dispersed $3CaO \cdot Al_2O_3 \cdot 6H_2O$ and a hydrated ferro oxide. The depth of reaction is dependent chiefly on the grain size and composition of the cement and the action is accelerated by increases in temperature. On continued hardening, further reactions may occur, as the formation of hydrated tetracalcium aluminate.

L. Forsen, Skanska Cementaktiebolaget, Limhamn, Sweden, "The Chemistry of Retarders and Accelerators." In developing a theory to account for the set of cements, Dr. Forsen reviewed published papers and presented many new data both on the reactions of the cement compounds and of cements with water and their behavior in the presence of gypsum.

N. Sundius, Sveriges Geologiska Undersokning, Stockholm, Sweden: "Constitution of Aluminous Cement Clinker." Dr. Sundius considered the chemical composition of the alumina cements, the compounds individually, and the compound contents of commercial clinkers concluding with a discussion of results of X-ray examinations and the crystallization process.

G. Assarsson, Sveriges Geologiska, Undersokning, Stockholm, Sweden: "Reactions of Aluminous Cement with Water." Dr. Assarsson first reviewed the literature on the hydration reactions of the compounds of alumina cements with water. The compounds considered were $5CaO \cdot 3Al_2O_3$, $CaO \cdot Al_2O_3$, $3CaO \cdot 5Al_2O_3$, $2CaO \cdot Fe_2O_3$ and $2CaO \cdot Al_2O_3 \cdot SiO_2$. Following this he considered the hydration of alumina cements.

F. M. Lea, Building Research Station, Garston, Herts, England: "Chemistry of Pozzolanas." Introductory to his presentation, Dr. Lea defined pozzolanas as follows: "Pozzolanas are siliceous materials which, though not cementitious in themselves, contain constituents which at ordinary temperatures will combine with lime in the presence of water to form compounds which have a low solubility and possess cementing properties." Under this definition Lea divides pozzolanas into the following groups: volcanic tuffs, diatomaceous earths, certain other highly siliceous rocks (e.g. French Gaize), burned clays and shales, by-products hydrated silica (e.g. Sistoff). Dr. Lea then presented data on the composition of the natural pozzolanas and to discuss the theories that have been advanced to explain the origin of their activity.

S. Giertz-Hedstrom, Laboratoriechet, Ingeniorsvetenskapsakademien, Stockholm, Sweden: "Physical Structure of Hydrated Cements." The speaker stated that study of the physical structure is an important supplement to purely chemical investigations

and constitutes the basis for an understanding of the function of cement and its application in concrete. In the system cement-water the cement particles form a first structure A, within which the hydration reactions take place and the new structure, B, is built up simultaneously with the breaking down and consumption of A. The rate of hydration and the volume changes accompanying it are in this connection of fundamental importance.

The physical structures of hardened cement can be studied in microscopic observations, and by means of microscopic examinations, X-ray analysis and measurement of the water fixation. The microscopic examinations give a good picture of the general development of the structure, but do not reveal the structure of the cement gel, the most important product of hydration. It is to be expected that X-ray analysis will in this connection be of great service later. The water fixation measurements constitute a very good indirect means of establishing the average structure.

The physical structure is determined by numerous factors, chiefly the kind of cement, but also in a very high degree by attendant circumstances such as water content, age, drying, etc. A complete discussion of these matters was not attempted, as this would mean dealing with the whole chemistry of cement and manufacture of concrete.

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Proposed Revisions of "Building Regulations for Reinforced Concrete"

(A. C. I. 501-36T)

Reported by Committee 501, Standard Building Code

A. W. STEPHENS, CHAIRMAN
R. R. ZIPPRODT, SECRETARY

The Tentative Building Regulations for Reinforced Concrete now in use were adopted as a Tentative Standard of the American Concrete Institute at the 32nd Annual Convention in Chicago, Feb. 25, 1936 and subsequently approved by letter ballot.

The four years which have intervened have again seen numerous advances in both design and construction. The revisions which Committee 501 is presenting at this time are the results of the joint efforts of this Committee and the Committee on Engineering Practice of the Concrete Reinforcing Steel Institute. Presentation of this report, embodying the proposed revisions, marks the completion of several years of further intensive effort so to revise the Building Regulations for Reinforced Concrete that they will be in agreement with current practice and developments.

While the proposed revisions follow the form of the 1936 Tentative Standard, many changes in the requirements have been made, among which the following are the more important:

1. Editorial changes have been made which place the notations, applying to any particular Chapter, at the head of that Chapter.
2. Further revisions to the Chapter on Concrete Quality and Working Stresses in order to conform to the latest developments in present-day practice.
3. The limitation of 30,000 p.s.i. as the maximum allowable unit tensile stress in one-way slabs has been extended to include steel reinforcement not exceeding $\frac{3}{8}$ inches in diameter.

4. The allowable unit shearing stress in footings has been limited to a maximum of 75 p.s.i., irrespective of the strength of concrete which may be used.

5. Section 709—Floors with Supports on Four Sides, has been revised and simplified for more ready use by the designing engineer. Tables of design factors for such slabs have also been added as footnotes for the same purpose.

6. The Chapter on Flat Slabs has been quite completely revised in order to take advantage of the principles of continuity which are rapidly coming into general acceptance by designing engineers. Committee 501, during the last four years, has devoted more time to an intensive study of the revisions, as presented, than to any other chapter of the Code. The Committee believes that its recommendations in this matter are the best that can be presented in view of our present knowledge of continuous structures.

7. Chapter 12—Footings—has also been generally revised after an extensive study of means to improve present practice in the design of isolated square or rectangular footings.

As presented for consideration, the revisions in Chapters 1, 2, 3 except Section 306), 4, 5, 6, 7 (except Section 709), 8, and 9, may be considered to be largely of editorial nature, having been referred to the Editorial Sub-Committee with power to act. Changes in chapters 10, 11 and 12 may be considered major revisions, and have first had majority approval of the respective Sub-Committees in charge before submission to the two main cooperating Committees.

Revisions of the following have been submitted to letter ballot of Committee 501, consisting in the main of 26 members; the number of members who returned ballots, and their vote affirmatively or negatively, is recorded herewith.

Chapter or Section	Total Committee Membership	Ballots Returned	Affirmative Vote	Negative Vote
Chapters 2, 3 & 4	27	23	21	2
Section 306	26	17	12	5
Section 709	Approved by Committee in New York in Feb., 1937, with *one* dissenting vote			
Section 904 (a) (4)	26	16	14	2
Chapter 10	26	17	14	3
Chapter 11	27	13	13	0
Chapter 12	26	16	16	0

Committee 501 submits this report with the recommendation that the revised Building Regulations for Reinforced Concrete be adopted as a Standard of the American Concrete Institute.

All items preceded by an asterisk (*) appear also in Errata—Building Regulations for Reinforced Concrete, see pp. 261 to 264. Such editorial and typographical corrections are now in effect for the Tentative Code of 1936, without reference to convention action on other revisions here reported.
—EDITOR

CHAPTER 1—GENERAL

Section 102(a)

*In second line, change "sizes" to "*size.*"

Precede last sentence in this section by the following: "*All plans submitted for approval or use on the work shall clearly show the strength of concrete for which all parts of the structure were designed.*"

Section 103(a)

*In the second and third lines, change the sentence to read: "and the design of which is either in conflict with, *or not covered by* these regulations."

Section 104

Change heading to "Definitions," omitting "and Notations."

104(a)

Omit the following definitions completely:

Aggregate
Column Strip
Middle Strip
Paneled Ceiling

Change the following definitions, to read as follows:

**Composite Column*—A column in which a steel or cast-iron section is completely encased in concrete containing *spiral and longitudinal reinforcement.*

**Deformed Bar*—In second and third lines *omit* the words "so as to firmly engage the surrounding concrete."

**Diagonal Band*—A group of *reinforcing* bars covering a width approximately 0.4 the average span, *placed symmetrically* with respect to the diagonal running from corner to corner of the panel of a flat slab.

**Direct Band*—A group of *reinforcing* bars, covering a width approximately0.4 l_1, *placed symmetrically* with respect to the center lines of the supporting columns of a flat slab.

**Drop Panel*—Change "Dropped" to "*Drop.*" The structural portion of a flat slab which is thickened *in the* area surrounding the column capital.

**Effective Area of Concrete*—The area of a section which lies between the centroid of the tensile reinforcement and the compression face of *the structural member.*

**Effective Area of Reinforcement*—In the second line omit the word "metal"; change the last line to read: "direction and *the direction* for which the effectiveness is to be determined."

**Plain Concrete*—In the first line, omit "metal."

**Ratio of Reinforcement*—Change to read: "The ratio of the effective area of the reinforcement to the effective area of the concrete at *any* section *of a structural member.*"

**Reinforced Concrete*—In the first line, change "metal" to "*reinforcement.*"

Section 104(b) Notations—to be completely omitted, since the special terms will now appear at the head of each chapter where they are specifically mentioned.

CHAPTER 2—MATERIALS AND TESTS

New Section 200—Notations

D = Deflection of a floor member under test load.
L = Span of member under load test. (See Sec. 202.).
t = Total thickness or depth of a member under load test.

**Section 204(a)*—Changes to be made as follows:

In 2nd line change "C9-30" to "C9-38"; also change "Tentative Specifications for High-Early Strength Portland Cement" to "Standard Specifications for High-Early Strength Portland Cement"; also change "C74-30T" to "C74-39."

Section 205(a)—Change the section to read:

Concrete aggregates shall conform to the "Standard Specifications for Concrete Aggregates (A. S. T. M. Serial Designation: C33-39), except that aggregates which have been shown by test to produce concrete of the required strength, durability, water-tightness, fire resistance and wearing qualities may be used under Sec. 302(a), Method 2, where authorized by the Commissioner of Buildings.

Section 206(a)—Change the section to read:

Water used in mixing concrete shall be clean, and free from injurious amounts of oils, acids, alkalis, organic materials or other deleterious substances. Water normally used for drinking purposes is satisfactory.

**Section 207(a)*—Change the last line as follows by adding:

"Serial Designation: A16-35), or for Axle-Steel Concrete Reinforcement Bars (A. S. T. M. Serial Designation: A160-36)".

**Section 207(b)*—Change the last line by adding:

"Concrete Reinforcement" (A. S. T. M. Serial Designation: A82-34), or for "Welded Steel Wire Fabric for Concrete Reinforcement" (A. S. T. M. Serial Designation: A185-37).

**Section 207(c)*—Change:

(A. S. T. M. Serial Designation: A9-34) to (A. S. T. M. Serial Designation: A9-36).

**Section 208(a)*—In the 1st line delete the words "at the work."

CHAPTER 3—CONCRETE QUALITY AND WORKING STRESSES

New Section 300—Notations

d = Depth from compression face of beam or slab to center of longitudinal tensile reinforcement.
f_c = Compressive unit stress in extreme fiber of concrete in flexure.
f'_c = Ultimate compressive strength of concrete *usually* at age of 28 days. (Sec. 301 (*a*)).
f_r = Permissible unit stress in the metal core of a composite column.
f'_r = Permissible unit stress on unencased steel columns and pipe columns.
f_s = Tensile unit stress in longitudinal reinforcement.
f_v = Tensile unit stress in web reinforcement.
n = Ratio of modulus of elasticity of steel to that of concrete, assumed in Sec. 305 (a) as equal to $\frac{30{,}000}{f'_c}$
r_a = Permissible unit working stress in concrete over the loaded area of a pedestal, pier, or footing.
u = Bond stress per unit of surface area of bar.
v = Shearing unit stress.
v_c = Shearing unit stress permitted on the concrete of the web.

**Section 301* (*b*)—Change footnote to read:

*In climates where frost action is not severe this section should be omitted.

Section 302 (*a*)—*Method 2—Controlled Concrete*

Change the 10th to 17th lines as follows: "The curve shall be established by at least three points, each point representing average values from at least four test specimens. The water-content used in the concrete for the structure as determined from the curve, shall correspond to a strength which is fifteen per cent greater than that called for on the plans. No substitutions shall be made in the materials used on the work without additional tests in accordance herewith to show that the quality of the concrete is satisfactory."

Section 303 (*b*)

Omit the present paragraph and substitute the following in its place: "(b) Laboratory control tests and field control tests, or laboratory control tests only, may be required. For each type of test required, there shall be at least one specimen for each 250 cu. yd. of concrete or fraction thereof on the job, and there shall be at least three specimens for each strength of concrete placed in any one day."

**Section 303* (*d*)

In 1st line, omit words "In all cases." In 2nd line, omit words, "shown by these tests." In 4th line, change "in the mix" to "in the mixture."

**Section 304* (*b*)

In 3rd line change "Tentative" to "Standard", also in 4th line, change "C94-35" to "C94-38."

**Section 305* (*a*)

In 2nd line change to read: "Where f'_c equals the *minimum specified* ultimate compressive strength at 28 days."

Section 305 (*a*)

For the table in the present Regulations substitute the revised Table A, p. 242.

Section 306 (*a*)

For the section in the present Regulations substitute the revised Section 306 (*a*).

305—Allowable Unit Stresses in Concrete

(*a*) The unit stresses in pounds per square inch on concrete to be used in the design shall not exceed the tabulated values (Table—Allowable Unit Stresses in Concrete) where f'_c equals the minimum specified ultimate compressive strength at 28 days.

306—Allowable Unit Stresses in Reinforcement

Unless otherwise provided in this Code, steel for concrete reinforcement shall not be stressed in excess of the following limits:

(*a*) *Tension*

(f_s = Tensile unit stress in longitudinal reinforcement)

(f_v = Tensile unit stress in web reinforcement)

20,000 p.s.i. for Rail-Steel Concrete Reinforcement Bars, Billet-Steel Concrete Reinforcement Bars (of intermediate and hard grades), Axle-Steel Concrete Reinforcement Bars (of intermediate and hard grades), and Cold-Drawn Steel Wire for Concrete Reinforcement.

18,000 p.s.i. for Billet-Steel Concrete Reinforcement Bars (of structural grade), and Axle-Steel Concrete Reinforcement Bars (of structural grade).

305—TABLE A—ALLOWABLE UNIT STRESSES IN CONCRETE

Description		Allowable Unit Stresses				
		For Any Strength of Concrete as Fixed by Test in Accordance with Sec. 302 $n = \frac{30000}{f'_c}$	When Strength of Concrete is Fixed by the Water-Content in Accordance with Sec. 302			
			f'_c = 2000 p. s. i. n = 15	f'_c = 2500 p. s. i. n = 12	f'_c = 3000 p. s. i. n = 10	f'_c = 3750 p. s. i. n = 8
Flexure: f_c						
Extreme fiber stress in compression	f_c	$0.40f'_c$	800	1000	1200	1500
Extreme fiber stress in compression adjacent to supports of continuous or fixed beams or of rigid frames	f_c	$0.45f'_c$	900	1125	1350	1688
Shear: v						
Beams with no web reinforcement and without special anchorage of longitudinal steel	v_c	$0.02f'_c$	40	50	60	75
Beams with no web reinforcement, but with special anchorage of longitudinal steel	v_c	$0.03f'_c$	60	75	90	113
Beams with properly designed web reinforcement but without special anchorage of longitudinal steel	v	$0.06f'_c$	120	150	180	225
Beams with properly designed web reinforcement and with special anchorage of longitudinal steel	v	$0.12f'_c$	240	300	360	450
*Flat slabs at distance d from edge of column capital or drop panel	v_c	$0.03f'_c$	60	75	90	113
**Footings	v_c		60	75	75	75
Bond: u						
‡In beams and slabs and one-way footings:						
Plain bars	u	$0.04f'_c$	80	100	120	150
Deformed bars	u	$0.05f'_c$	100	125	150	188
In two-way footings:						
Plain bars	u	$0.045f'_c$	90	113	135	169
Deformed bars	u	$0.056f'_c$	112	140	168	210
Bearing: f_c						
On full area	f_c	$0.25f'_c$	500	625	750	938
On one-third area†	f_c	$0.375f'_c$	750	938	1125	1405
Pedestals (see Sec. 1206)	r_a					

*See Sec. 807. **See Sec. 905(*a*).

†The allowable bearing stress on an area greater than one-third but less than the full area shall be interpolated between the values given.

‡Where special anchorage is provided (see Sec. 903(*a*), and 905(*a*)), one and one-half times these values in bond may be used in beams, slabs and one-way footings. The values given for two-way footings include an allowance for special anchorage.

(*b*) *Tension in One-Way Slabs*

For the main reinforcement, ⅜ inch or less in diamater, in one-way slabs, 50 per cent of the minimum yield point specified in the Standard Specifications of the American Society for Testing Materials for the particular kind and grade of reinforcement used, but in no case to exceed 30,000 p.s.i.

(*c*) *Compression, Vertical Column Reinforcement*

(f_s = Nominal working stress in vertical column reinforcement).

40 per cent of the minimum yield point specified in the Standard Specifications of the American Society for Testing Materials for the particular kind and grade of reinforcement used, but in no case to exceed 30,000 p.s.i. (See Sec. 1103*a* and 1103*d*).

(f_r = Allowable unit stress in the metal core of composite and combination columns):

Structural steel sections.....16,000 p.s.i.

Cast iron sections.....10,000 p.s.i.

Steel pipe.....See limitations of Sec. 1106 (*b*)

CHAPTER 4—MIXING AND PLACING CONCRETE

**Section 401* (a)—In 2nd line change "from the places" to "from the spaces."

Section 402 (a)—Add *new* section to read:

"(a) Unless otherwise authorized by the Commissioner of Buildings, the mixing of concrete shall be done in a batch mixer of approved type."

Change present (a) to (b).

Change present (b) to (c).

Change present (c) to (d).

**Section 402* (d)—Change "Tentative" to "Standard," and "C94-35" to "C94-38."

Section 404 (c)—Omit the present section and substitute therefor the following:

"All concrete shall be thoroughly compacted by suitable means during the operation of placing, and shall be thoroughly worked around the reinforcement, embedded fixtures, and into the corners of the forms. Vibrators may be used to aid in the placement of the concrete provided they are used under experienced supervision, and the forms are designed to withstand their action."

Section 405 (a)

In 3rd line change "provided" to "maintained;" in 4th line omit "of the seven-day period"

Section 406 (b)

In 5th line, change wording as follows: "and adequate means shall be provided for maintaining it at a temperature"

CHAPTER 5—FORMS AND DETAILS OF CONSTRUCTION

Section 502 (a)—At the end of this section add the following sentence:

"The results of suitable control tests may be used as evidence that the concrete has attained such sufficient strength."

**Section 505* (a)

In the 5th line change "The minimum clear spacing" to "minimum clear *distance.*"

**Section 507* (a)

Revise the second sentence in this section to read: "If concrete surfaces after removal of the forms are to be exposed to the weather or be in contact with the ground, the reinforcement shall be protected with not less than 2 inches of concrete."

**Section 507* (b)

Revise the first sentence of this section to read: "The concrete protective covering for reinforcement at surfaces not exposed directly to the ground or weather shall be not less than ¾ inches for slabs and walls; and not less than 1½ inches for beams girders and columns."

Section 507 (c)

Omit the first sentence.

Revise the second sentence to read: "If the code of which these regulations form a part specifies, as fire-protective covering of the reinforcement, thicknesses of concrete greater than those given in this section, then such greater thicknesses shall be used."

CHAPTER 6—DESIGN—GENERAL CONSIDERATIONS

New Section 600—Notations

f'_c = Ultimate compressive strength of concrete *usually* at age of 28 days. (Sec. 301 (a)).

n = Ratio of modulus of elasticity of steel to that of concrete = $\frac{E_s}{E_c}$; assumed in 305 (*a*) as equal to $\frac{30{,}000}{f'_c}$.

**Section 601* (*a*) (*2*)—In second line change "shall be taken" to "shall be *assumed.*"

**Section 603* (*c*)

On 3rd line change "provided in Sections 305 and 306" to "provided in Sections 305, 306 *and 1110.*"

CHAPTER 7—FLEXURAL COMPUTATIONS

New Section 700—Notations

A = Span length between opposite supports in one direction.

b = Width of rectangular beam or width of flange of T-beam.

b' = Thickness of web in beams of I or T sections.

B = Span length at right angles to A.

d = Depth from compression face of beam or slab to center of longitudinal tensile reinforcement; the diameter of a round bar or side of a square bar.

e_A = *Factor modifying r_A, used in obtaining an equivalent uniform load for bending moments on Span A.*

e_B = *Factor modifying r_B, used in obtaining an equivalent uniform load for bending moments on Span B.*

F_AA = *The distance between lines of inflection in span A, considering span A only to be loaded.*

F_BB = *The distance between lines of inflection in span B, considering span B only to be loaded.*

F_A = Ratio of the distance between assumed inflection points of the span A to span A in an isolated strip extending the entire width of the structure when a uniformly distributed load is applied to span A only.

F_B = Ratio as defined above, but applying to Span B.

K_A = Stiffness factor $\left(\frac{I}{A}\right)$ for span A of panel AB^1.

K_B = Stiffness factor $\left(\frac{I}{B}\right)$ for span B of panel AB^1.

K_{AR} = Stiffness factors *for any span adjacent to and continuous with span A.*

K_{BR} = Stiffness factors *for any span adjacent to and continuous with span B.*

l' = Clear span for positive moment and the average of the two adjacent clear spans for negative moment (see Sec. 701).

N = The sum of the lengths of those edges of panel AB *which are also edges of adjacent panels continuous with AB.*

$q_A = 6r_A(1 - e_A)$

$q_B = 6r_B(1 - e_B)$

r_A = *Proportion of the total load carried by Span A of slab.*

r_B = *Proportion of the total load carried by Span B of slab.*

t_1 = *Minimum* total thickness of slab.

w = Uniformly distributed load per unit of length of beam or slab.

x = *Distance from face of support to point in span.*

***Section 702 (b) (4)*

Change 3rd line to read: "For structural, intermediate and hard grade, and rail *and axle* steel..... .0025 *bd*"

Section 707—Change section number to *706*.

Section 706—Change section number to *707*.

***Section 707*—Change heading to: "*Concrete Joist* Floor Construction."

***Section 707 (a)*

Change section to read: "*Concrete joist* floor construction consists of concrete *joists* and slabs placed monolithically with or without burned clay or concrete tile fillers. The *joists* shall not be farther apart than 30 inches face to face. The *joists* shall be straight, not less than 4 inches wide, nor of a depth more than 3 times the width."

***Section 707 (b)*

In second line change "in the ribs" to "in the *joists*"

In fourth line change "with the ribs" to "with the *joists*."

***Section 707 (c)*

In second line change "distance between ribs" to "distance between *joists*."

***Section 707 (d)*

In third line change "ribs" to "*joists*".

In fourth line change "to the ribs" to "to the *joists*."

Section 709 (1) (2)

Remove present footnote (2).

***Section 709 (a)*

In 2nd, 3rd and 4th lines change "ribs" to "*joists*."

Remove completely the last part of the first sentence as follows: "and concrete ribs with top slabs placed monolithically with the ribs."

Section 709 (b) (c) and (d)—Omit completely.

New Section 709 (b)—Minimum Slab Thickness

The slab thickness shall satisfy prescribed working stresses and shall be not less than 4 inches nor less than

$$t_1 = \frac{A + B - 0.1N}{72} \quad \sqrt[3]{\frac{2000}{f_c'}}$$

New Section 709 (c)—Bending Moments and Shears

The bending moment at any section shall be determined with the same coefficients prescribed for one way construction (Section 701, footnote), using the following equivalent uniform load per unit length of span considered:

Slab: Strip of unit width, span A, $(e_A r_A)w$

Beam: Span A, carrying one half of load from panel width B, $\frac{B}{2}(1 - e_A r_A)w$

The shear at any section distance x from the face of the support shall be taken as:

Slab: Strip of unit width, span A, $\left(\frac{A}{2} - x\right)\left((r_A - \frac{q_A x}{A}\right)w$

Beam: Span A, carrying one half of load from panel, width B,

$$\frac{B}{2}\left(\frac{A}{2} - x\right)\left(1 - r_A + \frac{q_A x}{A}\right)w$$

For span B substitute above A for B, B for A, e_B for e_A, r_B for r_A, and q_B for q_A.

The factors e_A, r_A, etc., may be taken from Table II, footnote (2) below after the ratio F_AA/F_BB or F_BB/F_AA on which they depend has been determined by the aid of Table 1, footnoote (1); or the several factors may be computed from the formulas which appear in the footnote (2).

Footnotes:

(1) For comparative use the moment of inertia of a slab shall be taken as that of the total plain concrete section.

(2) Formulas for F_A, F_B, e_A, e_B, r_A, r_B. (See "Slabs Supported on Four Sides" by J. Di Stasio and R. P. Van Buren, JOURNAL of the A. C. I., January-February, 1936).

End Span, continuous at one end only

$$F_A = 1 - \frac{0.25}{1 + \frac{7K_A}{8K_{AR}}}$$

Interior continuous span with K_{AR} the same for both adjacent spans continuous with A

$$F_A = \sqrt{1 - \frac{1}{1.5 + \frac{7K_A}{8K_{AR}}}}$$

For interior spans where the spans adjacent to and in continuation of the span A under consideration differ in stiffness, for F_A use the average of the two values, one obtained using K_{AR} for the span in continuation on one end of the span A, and the other obtained by using the value of K_{AR} for the span at the other end.

To obtain F_B replace K_A with K_B and K_{AR} with K_{BR}.

$$r_A = \frac{1}{1 + \left(\frac{F_AA}{F_BB}\right)^3} = 1 - r_B$$

$$e_A = \frac{2}{4 - \frac{F_BB}{F_AA}} \qquad e_B = \frac{2}{4 - \frac{F_AA}{F_BB}}$$

$= 1.00$ for $F_BB/F_AA = 2$ $\qquad e_B = \frac{1}{2}$ as $F_AA/F_BB \doteq 0$

The total load carried by a strip of slab of unit width, span A, equals r_AwA and is considered to vary in intensity from $r_Aw(3e_A - 2)$ at the center of the span, to $r_Aw(4 - 3e_A)$ at the supports.

The total load carried by a beam of span A, one-half panel tributary width, equals $(1 - r_A)\frac{wBA}{2}$ and varies uniformly in intensity from $(1 + 2r_A - 3e_Ar_A)\frac{wB}{2}$ at the center of the span to $(1 - 4r_A + 3e_Ar_A)\frac{wB}{2}$ at the supports.

When considering the B spans use the above expressions, replacing A with B, B with A, r_A with r_B, and e_A with e_B.

TABLE I. F_A and F_B

The values given in the table are for F_A directly. They are also the values for F_B when the designation K_A/K_{AR} is replaced by K_B/K_{BR}.

Span A	$\frac{K_A}{K_{AR}}$ =	0.00	0.25	0.50	0.67	0.80	1.00	1.25	1.50	2.00	4.00	0, ∞
Interior*	F_A =	0.58	0.65	0.69	0.72	0.74	0.76	0.78	0.80	0.83	0.89	1.00
End	F_A =	0.75	0.80	0.83	0.84	0.85	0.87	0.88	0.89	0.91	0.95	1.00
Simple	F_A =	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

*For interior spans where the spans adjacent to and in continuation of the span A under consideration differ in stiffness, for F_A use the average of the two values, one obtained using K_{AR} for the span in continuation on one end of the span A, and the other obtained by using the value of K_{AR} for the span at the other end.

For values of K_A/K_{AR} between 2/3 and 3/2 the value of F_A may be taken as 0.76 for interior spans and 0.87 for end spans. (Footnote (2) continued on next page).

New Section 709 (d)—Arrangement of Reinforcement

(1) In any panel, the reinforcement per unit width in the long direction shall be at least one-third of that provided in the short direction.

(2) The positive moment reinforcement adjacent to a continuous edge only and for a width not exceeding one-fourth of the shorter dimension of the panel may be reduced 25 per cent.

(3) At a non-continuous edge negative moment reinforcement per unit width in amount at least as great as one-half of that required for maximum positive moment for the center one-half of the panel shall be provided across the entire width of the exterior support.

(4) The spacing of the reinforcement shall be not more than 3 times the slab thickness and the ratio of reinforcement shall be at least 0.0025.

(5) The amount of reinforcement in any one foot wide strip shall be at least 0.003 times the product of the width of strip by the effective depth.

Present Sections (*e*), (*f*), (*g*), (*h*), (*i*) and (*j*) to be completely omitted.

**Section 710* (*a*)

Change, in first line, "ribbed floor" to "*concrete joist floor.*"

CHAPTER 8—SHEAR AND DIAGONAL TENSION

New Section 800—Notations

A_v = Total area of web reinforcement in tension within a distance of s (*measured in a direction parallel to that of the main reinforcement*), or the total area of all bars bent up in any one plane.

α = Angle between inclined web bars and axis of beam.

b = Width of rectangular beam or width of flange of T-beam.

b' = Thickness of web in beams of I or T sections.

Footnote (2) Sec. 709c continued.

TABLE II

The value of e_A or e_B shall be taken as unity for the computation of shear and bending moment in slabs and beams where the span in direction under consideration is not rigidly attached to the supports at one or both ends of the span.

$\frac{F_A A}{F_B B}$	r_A or $1 - r_B$	e_A	$e_A r_A$	$1 - e_A r_A$	$q_A = 6r_A(1 - e_A)$
0.00	1.00	1.00	1.00	0.00	0.00
0.50	0.89	1.00	0.89	0.11	0.00
0.55	0.86	0.92	0.79	0.21	0.41
0.60	0.82	0.86	0.71	0.29	0.69
0.65	0.78	0.81	0.63	0.37	0.89
0.70	0.74	0.78	0.58	0.42	0.98
0.80	0.66	0.73	0.48	0.52	1.07
0.90	0.58	0.69	0.40	0.60	1.08
1.00	0.50	0.67	0.33	0.67	1.00
1.10	0.43	0.65	0.28	0.72	0.90
1.20	0.37	0.63	0.23	0.77	0.82
1.30	0.31	0.62	0.19	0.81	0.71
1.40	0.27	0.61	0.16	0.84	0.63
1.50	0.23	0.60	0.14	0.86	0.55
1.60	0.20	0.59	0.12	0.88	0.49
1.80	0.15	0.58	0.09	0.91	0.38
2.00	0.11	0.57	0.06	0.94	0.28
3.00	0.04	0.55	0.02	0.98	0.11
$\frac{F_B B}{F_A A}$	r_B or $1 - r_A$	e_B	$e_B r_B$	$1 - e_B r_B$	$6r_B(1 - e_B) = q_B$

d = Depth from compression face of beam or slab to center of longitudinal tensile reinforcement.
f_c' = Ultimate compressive strength of concrete *usually* at age of 28 days.
f_v = Tensile unit stress in web reinforcement.
j = Ratio of distance between centroid of compression and centroid of tension to the depth (d).
s = Spacing of stirrups or of bent bars in a direction parallel to that of the main reinforcement.
t_2 = Thickness of flat slab without drop panels.
t_3 = Thickness of flat slab (with drop panels) at points outside the drop panel.
V = Total shear.
V' = Excess of the total shear over that permitted on the concrete.

**Section 801* (*a*)—Revise to read as follows:

The shearing unit stress (v), *as a measure of diagonal tension*, in reinforced concrete flexural members shall be computed by formula (14).

**Section 801* (*c*)—Revise first line to read as follows:

In *concrete joist floor* construction, where burned clay or concrete tile are used, etc.

Section 807—Wherever they appear in this section, change t_1 to t_2, and t_2 to t_3.

**Section 807* (*a*) (1)—Change to read:

(1) 0.03 f'_c, when at least 50 per cent of the total negative reinforcement *in the column strip* passes directly over the column capital.

(2) 0.025 f'_c, when 25 per cent of the total negative reinforcement *in the column strip* passes directly over the column capital.

Section 808 (*a*)—Revise to read as follows:

In isolated footings the shearing unit stress computed by formula (14) on the critical section (see *1205* (*a*)), shall not exceed *0.03f_c', nor in any case shall it exceed 75 p.s.i.*

CHAPTER 9—BOND AND ANCHORAGE

New Section 900—Notations

d = Depth from compression face of beam or slab to center of longitudinal tensile reinforcement.
f'_c = Ultimate compressive strength of concrete *usually* at age of 28 days.
j = Ratio of distance between centroid of compression and centroid of tension to the depth (d).
Σ_o = Sum of perimeters of bars in one set.
u = Bond stress per unit of surface area of bar.
V = Total shear.

Section 904 (*a*) (3)—Change to read as follows:

"Embedment above the mid-depth of the beam a distance sufficient to develop the stress to which the bar will be subjected at a bond stress of not to exceed .04f'_c on plain bars nor .05f'_c on deformed bars."

Section 904 (*a*) (4)—Change to read as follows:

"Standard hook, considered as developing 10,000 p.s.i., plus embedments sufficient to develop by bond the remainder of the stress to which the bar is subjected. The unit bond stress shall not exceed that specified elsewhere in Section 305. The effective embedded length shall not be taken as exceeding the distance between the mid-depth of the beam and the tangent point of the hook."

Section 905 (*a*)—Change first sentence to read as follows:

All bars in footing slabs shall be anchored by means of standard hooks.

CHAPTER 10—FLAT SLABS—TWO-WAY AND FOUR-WAY SYSTEMS WITH SQUARE OR RECTANGULAR PANELS

New Section 1000—Notations

A = *The distance from the center of the column in a flat slab to its face, or to the edge of the 45° pyramid enclosed within the column capital.*

b = *Minimum side or diameter of a drop panel in a flat slab.*

c = Diameter, in feet, of column capital of a flat slab at the underside of the slab, or *drop* panel. No portion of the column capital shall be considered for structural purposes which lies outside the largest 90° cone that can be included within the outlines of the column capital.

d = The diameter of a round bar or side of a square bar.

f'_c = Ultimate compressive strength of concrete *usually* at age of 28 days.

I = Moment of inertia of a section about the neutral axis for bending.

l = Span length of a flat slab (usually expressed in feet) center to center of columns in the direction in which moments are considered.

M_c = *Total moment in the column strip of a flat slab.*

M_m = *Total moment in the middle strip of a flat slab.*

M_o = Sum of positive and negative bending moments at the principal design sections of a panel of a flat slab.

R = (*See formula in Section 1002 (b) 4.*)

$$\frac{E_c I_c}{h} = \frac{\textit{Modulus of elasticity of concrete by moment of inertia}}{\textit{height}}$$

for lower column.

$$\frac{E'_c I'_c}{h'} = \frac{\textit{Modulus of elasticity of concrete by moment of inertia}}{\textit{height}}$$

for upper column.

$$\frac{E_s I_s}{l} = \frac{\textit{Modulus of elasticity of slab concrete by moment of inertia of slab}}{\textit{span}}$$

for the slab.

W = Total dead and live load uniformly distributed over a single panel area.

x = (See formula in Section 1002 (*a*) (7))

For sections 1001 to 1012 inclusive in present regulations substitute new Sections 1001 to 1008 inclusive as follows:

1001—Scope

(*a*) The term flat slab shall mean a reinforced concrete slab supported by columns with or without flaring heads or column capitals, with or without depressed or drop panels and generally without beams or girders.

(*b*) Recesses or pockets in flat slab ceilings, located between widely spaced reinforcing bars and forming cellular or two-way ribbed ceilings, whether left open or filled with permanent fillers, shall not prevent a slab from being considered a flat slab; but allowable unit stresses shall not be exceeded.

1002—Determination of Bending and Shear

(*a*) Except in the special cases of flat slab construction where specified coefficients for bending may be used as provided in paragraph (*b*) of this Section, bending and shear in flat slabs and their supports shall be determined by the application of the principles of continuity and all sections shall be proportioned to resist the moments and shears thus obtained. In the analysis of flat slabs as continuous frames the

following simplifying assumptions, or others more nearly in accordance with fact, may be made:

(1) The structure may be considered divided into a number of bents, each consisting of a row of columns and strips of supported slabs, each strip bounded laterally by the center line of the panel either side of the row of columns. The bents shall be taken longitudinally and transversely of the building.

(2) Each such bent may be analyzed in its entirety; or each floor thereof and the roof may be analyzed separately with its adjacent columns above and below, the columns being assumed fixed at their remote ends. Where slabs are thus analyzed separately, in bents more than four panels long, it may be assumed in determining the bending at a given support that the slab is fixed at any support two panels distant therefrom beyond which the slab continues.

(3) The joints between columns and slabs shall be considered rigid and this rigidity may be assumed to extend in the slabs to the edge of a forty-five degree pyramid enclosed within the concrete capital or bracket, and in the column to the intersection of the sides of such pyramid and column. The change in length of columns and slabs due to direct stress, and deflections due to shear, may be neglected. Where metal column capitals are used, account shall be taken of their contributions to stiffness and resistance to bending and shear.

(4) The supports of columns may be assumed free from settlement or movement unless the amount thereof can be reasonably determined.

(5) The moment of inertia of slab or column at any cross-section may be assumed to be that of the gross section of the concrete. Variation in the moments of inertia of the slabs and columns along their axes shall be taken into account.

(6) Where the load to be supported is definitely known, the structure shall be analyzed for that load. Where the live load is variable but does not exceed three-quarters of the dead load, or the nature of the live load is such that all panels will be loaded simultaneously, the maximum bending may be assumed to obtain at all sections under full live load. Elsewhere, maximum positive bending near mid-span of a panel may be assumed to obtain under full live load in the panel and in alternative panels; and maximum negative bending at a support may be assumed to obtain under full live load in the adjacent panels only.

(7) For slabs in which neither beams nor girders contribute to the transfer of the slab loading to the columns supporting it, the maximum negative moment for which the slab shall be proportioned need not be greater than that prevailing at a distance xl from the column center, where

$$x = .50 - \sqrt{.183 - .45\frac{A}{l}}$$

in which A is the distance from the center of the column to its face or to the edge of the forty-five degree pyramid enclosed within the column capital.

For slabs bearing on beams, girders or walls, the maximum negative moment across the supporting members for which the slab is proportioned shall be that prevailing at the face of the support.

(8) The bending at critical sections across the slabs of each bent may be apportioned between the column strip and middle strip, as defined in Section *1003*, in the ratio of the specified coefficients which effect such apportionment in the special cases of flat slabs provided for in paragraph (*b*) of this section.

(9) The maximum bending in columns may be assumed to obtain under full live load in alternate panels. Columns shall be proportioned to resist the maximum bending combined with the maximum direct load consistent therewith, and for maximum direct load combined with the bending under full load, the direct load subject to allowed reductions, in the manner provided in Chapter 11.

(*b*) In the special cases of flat slabs in which

(1) The ratio of length to width of panel does not exceed 1.33, and

(2) The slab is continuous for at least three panels in each direction, and

(3) The successive spans in each direction are within 10 per cent of each other and

(4) The stiffness ratio of columns to slab $R = \dfrac{\dfrac{E_c I_c}{h} + \dfrac{E'_c I'_c}{h'}}{\dfrac{E_s I_s}{l}}$

and the ratio of live load to dead load comply with any one of the four following classifications:

Class A. The Ratio R for all exterior and interior supports is at least 10.

Class B. The Ratio R for all exterior supports is at least 10, that of interior supports any value and the live to dead load ratio is at most 0.75.

Class C. The Ratio R for all interior supports is at least 10, that of the exterior support equal to 1.

Class D. The Ratio R for the exterior supports equals 0.10, that of the interior supports any value and the live to dead load ratio is at most 0.75,—in such special cases the bending moments at critical sections may be determined by use of specified coefficients as provided in the following sections.

1003—Panel Strips and Critical Design Sections

(*a*) A flat slab panel shall be considered as consisting of strips in each direction as follows:

A middle strip one half panel in width, symmetrical about panel center line and extending through the panel in the direction of the span for bending.

A column strip consisting of the two adjacent quarter-panels either side of the column center lines.

(*b*) The critical sections for bending are located as follows:

Sections for negative bending shall be taken along the edges of the panel, on column center lines between capitals and around the perimeters of column capitals.

Sections for positive bending shall be taken on the center lines of the panels, at mid-span of the strips.

(*c*) Only the reinforcement which crosses a critical section within a strip may be considered effective to resist bending in the strip at that section. Reinforcement which crosses such section at an angle with the centerline of the strip shall be assumed to contribute to the resistance of bending only its effective area in the direction of the strip, as defined in Chapter 1.

(*d*) The width of the strip shall be taken as the width of the section in computing compression due to bending, except that the width of the drop panel shall be taken for negative bending on a section through a drop panel. Account shall be taken of any recesses which reduce the compressive area.

1004—Slab Thickness and Drop Panels

(*a*) The thickness of a flat slab and the size and thickness of the drop panel, where used, shall be such that the compressive stress due to bending at the critical sections of any strip and the shear about the column capital and the drop panel shall not exceed the unit stresses allowed in concrete of the quality used.

(*b*) Slab thickness shall not, however, be less than $\frac{l}{40}\sqrt[3]{\frac{2000}{f'_c}}$

(*c*) The thickness of the drop panel below the slab shall not be more than one-fourth the distance from the edge of the column capital to the edge of the drop panel.

1005—Bending Moments in Interior and Exterior Panels

(*a*) The numerical sum of the positive and negative bending moments in the direction of either side of an interior rectangular panel shall be assumed as not less than

$$M_o = 0.09\ Wl\left(1 - \frac{2c}{3l}\right)^2$$

In the special case where $c = 0.225l$ this value becomes $M_o = 0.065\ Wl$.

(*b*) The bending moments for design at the critical sections of column and middle strips in interior and exterior bents of flat slab, with or without drop panels, shall be assumed as given in Table 1 or 2 respectively for the appropriate classifications of column to slab stiffness Ratio R and live to dead load ratio.

(*c*) In the case of end or side panels when the slab is not continuous at one edge or two adjacent edges, the negative moment over the exterior supports shall be taken from Tables 1 or 2 under the heading "Flexible Edge" when a marginal beam with depth 1½ times the slab thickness or less is provided, and under the heading "Rigid Edge" when a marginal beam with depth greater than 1½ times the slab thickness or a wall supports the slab.

(*d*) When there is a beam or wall at the center line of columns in the interior portion of a continuous flat slab, the negative moments over the beam or wall for which the slab shall be proportioned shall be taken from Table 1 or 2 under the heading "Rigid Edge" for classification *A* or *B*.

(*e*) When the slab is supported along the center lines of exterior or interior columns by rigid beams or walls, the half column strips adjacent and parallel thereto may be designed for one quarter of the full column strip moments given in Tables 1 and 2.

(*f*) Rigid beams along the center line of exterior or interior columns shall be designed to carry, in addition to any special load, one quarter of the total dead and live load on the adjacent panels.

(*g*) Slabs supported by rigid beams or walls along both lines of supports shall be designed as one or two way slabs.

(*h*) For design purposes any of the moment coefficients of Tables 1 and 2 may be varied by not more than six per cent, but the numerical sum of the positive and the negative moments in a panel shall not be taken as less than the total amounts specified.

1006—Capitals and Brackets

(*a*) Where a column is without a flaring concrete capital the distance (*c*) shall be taken as the diameter of the column. Structural metal embedded in the slab or drop panel may be regarded as contributing to resistance in bending and shear.

(*b*) Where a reinforced concrete beam frames into a column without capital or bracket on the same side with the beam, the value of (*c*) may be taken as the width of the column plus twice the projection of the beam above or below the slab or drop panel for computing bending in strips parallel to the beam.

(*c*) Brackets capable of transmitting the negative bending and the shear in the column strips to the columns without excessive unit stress may be substituted for column capitals at exterior columns. The value of (*c*) where brackets are used shall be taken as twice the distance from the center of the column to a point where the bracket is 1½ inches thick, but not more than the thickness of the column plus twice the depth of the bracket.

(*d*) The average of the diameters (*c*) of the column capitals at the four corners of a panel shall be used in determining the bending in the middle strips of the panel. The average of the diameters (*c*) of the two column capitals at the ends of a column strip shall be used in determining bending in the strip.

1007—Arrangement of Reinforcement

(*a*) Slab reinforcement shall be provided to resist the bending not only at critical sections, but also at intermediate sections.

(*b*) Bars shall be spaced evenly across strips or bands and the spacing shall not exceed three times the slab thickness. In four-way arrangement of reinforcement the width of direct and middle strip negative bands shall be four tenths the panel width, and the width of diagonal bands shall be two tenths the sum of length and width of panel.

(*c*) The length of bars shall not be less than, and their points of bend shall be approximately as, provided in Table 3. (See next page.)

(*d*) In exterior panels the reinforcement perpendicular to the discontinuous edge for positive bending, shall extend to the edge and have embedment of at least six inches in spandrel beams or columns. All such reinforcement for negative bending shall be bent, hooked or otherwise anchored in spandrel beams or columns.

1008—Openings in Flat Slabs

Openings of any size may be cut through a flat slab if provision is made for the total positive and negative resisting moments, as required in Section 1005. without exceeding the allowable stresses as given in Section 807.

CHAPTER 11—REINFORCED CONCRETE COLUMNS AND WALLS

New Section 1100—Notations

A_c = Area of core of a spirally-reinforced column measured to the outside diameter of the spiral; net area of concrete section of a composite column.

A_g = The overall or gross area of spirally reinforced or tied columns; the total area of the concrete encasement of combination columns.

A_r = Area of the steel or cast-iron core of a composite column; the area of the steel core in a combination column.

A_s = Effective cross-sectional area of reinforcement in compression in columns.

$C = \frac{f_s}{0.4f'_c}$ (See Sec. 1110 (*a*))

c = Distance from gravity axis to extreme fiber in compression (in a column).

d = The least lateral dimension of a concrete column.

e = Eccentricity of the resultant load on a column, measured from the gravity axis.

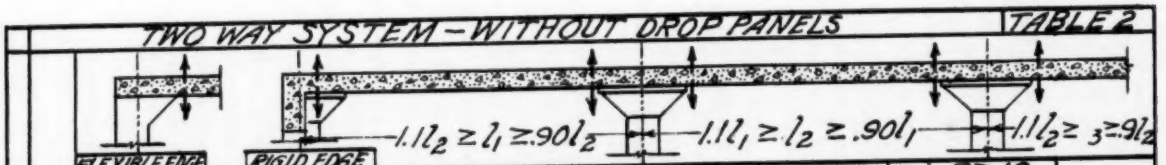
MOMENTS TO BE USED IN DESIGN OF INTERIOR AND EXTERIOR PANELS OF FLAT SLABS

Expressed in terms of "M_0" for variable "c" ¢ in terms of "wl^2" for constant "c" = .225 l for various combinations of "R" *

TWO WAY SYSTEM WITH DROP PANEL = .35l AND 1½ t — TABLE I

CLASSIFICATION	Bands	FLEXIBLE EDGE		RIGID EDGE												
		R ≥ 10		R ≥ 10				R ≥ 10				R ≥ 10				
A	Col.	.47M_0	.0306wl^2	−.56M_0	.0364wl^2	+.23M_0	.015wl^2	−.62M_0	.0403wl^2	+.22M_0	.0143wl^2	−.61M_0	.0396wl^2			
	Mid.	.11 "	.0072 "	−.56 "	.0364 "	+.18 "	.0117 "	−.18 "	.0117 "	+.17 "	.0111 "	−.18 "	.0117 "			
	Total	.58 "	.0378 "	−1.12 "	.0728 "	+.41 "	.0267 "	−.80 "	.0520 "	+.39 "	.0254 "	−.79 "	.0513 "	$\frac{L.L.}{D.L.}$ = any value		
		R ≥ 10		R ≥ 10				R = any value				R = any value				
B	Col.	.44M_0	.0286wl^2	−.52M_0	.0338wl^2	+.23M_0	.015wl^2	−.62M_0	.0403wl^2	+.22M_0	.0143wl^2	−.54M_0	.0351wl^2			
	Mid.	.10 "	.0065 "	−.52 "	.0338 "	+.18 "	.0117 "	−.18 "	.0117 "	+.17 "	.0111 "	−.17 "	.0111 "			
	Total	.54 "	.0351 "	−1.04 "	.0672 "	+.41 "	.0267 "	−.80 "	.0520 "	+.39 "	.0254 "	−.71 "	.0462 "	$\frac{L.L.}{D.L.} \leq .75$		
		R = 1		R = 1				R = 10				R = 10				
C	Col.	.24M_0	.0156wl^2	−.27M_0	.0176wl^2	+.36M_0	.0234wl^2	−.84M_0	.0546wl^2	+.22M_0	.0143wl^2	−.61M_0	.0396wl^2			
	Mid.	.06 "	.0039 "	−.27 "	.0176 "	+.24 "	.0156 "	−.25 "	.0162 "	+.17 "	.0111 "	−.18 "	.0117 "			
	Total	.30 "	.0195 "	−.54 "	.0352 "	+.60 "	.0390 "	−1.09 "	.0708 "	+.39 "	.0254 "	−.79 "	.0513 "	$\frac{L.L.}{D.L.}$ = any value		
		R = 0.10		R = 0.10				R = any value				R = any value				
D	Col.	.24M_0	.0156wl^2	−.15M_0	.00975wl^2	+.52M_0	.0338wl^2	−.87M_0	.0566wl^2	+.22M_0	.0143wl^2	−.54M_0	.0351wl^2			
	Mid.	.06 "	.0039 "	−.15 "	.00975 "	+.34 "	.0221 "	−.26 "	.0169 "	+.17 "	.0111 "	−.17 "	.0111 "			
	Total	.30 "	.0195 "	−.30 "	.0195 "	+.86 "	.0559 "	−1.13 "	.0735 "	+.39 "	.0254 "	−.71 "	.0462 "	$\frac{L.L.}{D.L.} \leq .75$		

TWO WAY SYSTEM — WITHOUT DROP PANELS — TABLE 2



		FLEXIBLE EDGE		RIGID EDGE											
		R ≥ 10		R ≥ 10				R ≥ 10				R ≥ 10			
A	Col.	$.39M_o$	$.0254wl^2$	$-.48M_o$	$.031wl^2$	$+.28M_o$	$.0182wl^2$	$-.50M_o$	$.0325wl^2$	$+.28M_o$	$.0182wl^2$	$-.50M_o$	$.0325wl^2$		
	Mid.	.10 "	.0065 "	-.48 "	.031 "	+.20 "	.0130 "	-.18 "	.0117 "	+.20 "	.0130 "	-.17 "	.0111 "	$\frac{L.L.}{D.L.}$ = any value	
	Total	.49 "	.0319 "	-.96 "	.062 "	+.48 "	.0312 "	-.68 "	.0442 "	+.48 "	.0312 "	-.67 "	.0436 "		
		R ≥ 10		R ≥ 10				R = any value				R = any value			
B	Col.	$.37M_o$	$.024wl^2$	$-.45M_o$	$.0292wl^2$	$+.28M_o$	$.0182wl^2$	$-.50M_o$	$.0325wl^2$	$+.28M_o$	$.0182wl^2$	$-.45M_o$	$.0292wl^2$		
	Mid.	.09 "	.006 "	-.45 "	.0292 "	+.20 "	.0130 "	-.18 "	.0117 "	+.20 "	.0130 "	-.16 "	.0104 "	$\frac{L.L.}{D.L.} \leq .75$	
	Total	.46 "	.030 "	-.90 "	.0584 "	+.48 "	.0312 "	-.68 "	.0442 "	+.48 "	.0312 "	-.61 "	.0396 "		
		R = 1		R = 1				R ≥ 10				R ≥ 10			
C	Col.	$.24M_o$	$.0156wl^2$	$-.27M_o$	$.0176wl^2$	$+.41M_o$	$.0261wl^2$	$-.68M_o$	$.0442wl^2$	$+.28M_o$	$.0182wl^2$	$-.50M_o$	$.0325wl^2$		
	Mid.	.06 "	.0039 "	-.27 "	.0176 "	+.29 "	.0189 "	-.24 "	.0156 "	+.20 "	.0130 "	-.17 "	.0111 "	$\frac{L.L.}{D.L.}$ = any value	
	Total	.30 "	.0195 "	-.54 "	.0352 "	+.70 "	.0456 "	-.92 "	.0598 "	+.48 "	.0312 "	-.67 "	.0436 "		
		R = 0.10		R = 0.10				R = any value				R = any value			
D	Col.	$.24M_o$	$.0156wl^2$	$-.15M_o$	$.00975wl^2$	$+.58M_o$	$.0377wl^2$	$-.71M_o$	$.0461wl^2$	$+.28M_o$	$.0182wl^2$	$-.45M_o$	$.0292wl^2$		
	Mid.	.06 "	.0039 "	-.15 "	.00975 "	+.42 "	.0273 "	-.25 "	.0163 "	+.20 "	.0130 "	-.16 "	.0104 "	$\frac{L.L.}{D.L.} \leq .75$	
	Total	.30 "	.0195 "	-.30 "	.0195 "	+1.00 "	.0650 "	-.96 "	.0624 "	+.48 "	.0312 "	-.61 "	.0396 "		

$$M_o = .09Wl\left(1 - \frac{2c}{3l}\right)^2 \qquad *R = \frac{\frac{E_c I_c}{h} + \frac{E_c' I_c'}{h}}{\frac{E_s I_s}{l}} = \frac{\text{stiffness factors of upper and lower column.}}{\text{stiffness factor of slab.}}$$

TABLE 3—LENGTH OF BARS AND POINTS OF BEND (SEC. 1007*c*)

	With Drop		Without Drop	
	General Case	$c = .225l$	General Case	$c = .225l$
TWO-WAY FLAT SLAB (COLUMN STRIP)				
Length of straight bars (not less than .4 of total band steel)	$l - b + (2'$ or $40d)$	$.65l + (2'$ or $40d)$	$.75l$	$.75l$
Length of bent bars (not less than .4 total band steel)	$1.5l + .6c$†	$1.635l$†	$1.44l + .66c$†	$1.59l$†
Length of additional straight bars over column head (if required)	$.5l + .6c$	$.635l$	$.44l + .66c$	$.59l$
Point of top bend in bent bars (from column centers)	$.25l$	$.25l$	$.25l$	$.25l$
MIDDLE STRIP				
Length of straight bars (not more than .5 total band steel)	$.65l$	$.65l$	$.7l$	$.7l$
Length of bent bars (not less than .5 total band steel)	$1.5l$†	$1.5l$†	$1.5l$†	$1.5l$†
Point of top bend in bent bars (from column centers)	$.175l$	$.175l$	$.15l$	$.15l$
FOUR-WAY FLAT SLAB COLUMN STRIP				
Length of straight bars (not less than .4 total band steel)	$l - b + (2'$ or $40d)$	$.65l + (2'$ or $40d)$	$.75l$	$.75l$
Length of bent bars (not less than .4 total band steel)	$1.5l + .6c$†	$1.635l$†	$1.44l + .66c$†	$1.59l$†
Length of additional straight bars over column head (if required)	$.5l + .6c$	$.635l$	$.44l + .66c$	$.59l$
Point of bend for bent bars (from column centers)	$.2l$	$.2l$	$.2l$	$.2l$
DIAGONAL BAND				
Length of straight bars (not more than .6 total band steel area)	$l - b + (2'$ or $40d)$	$.65l + (2'$ or $40d)$	$.75l$	$.75l$
Length of bent bars (not less than .4 total band steel area)	$2.21l$†	$2.21l$†	$2.21l$†	$2.21l$†
Point of bend for bent bars (from column centers)	$.33l$	$.33l$	$.33l$	$.33l$
Length of additional straight bars over column head (if required)	$.8l$	$.8l$	$.8l$	$.8l$
Top band across middle of direct band (length of straight bars)	$.5l$	$.5l$	$.5l$	$.5l$

†Note: To these lengths proper allowance to be added for bends.

$F = \dfrac{\text{Yield point of pipe}}{45{,}000}$ (See Sec. 1106 (*b*))

f'_c = Ultimate compressive strength of concrete *usually* at age of 28 days.

f_r = Permissible unit stress in the metal core of a composite column.

f'_r = Permissible unit stress on unencased steel columns and pipe columns.

f_s = Nominal working stress in vertical column reinforcement.

f'_s = Useful limit stress of spiral reinforcement.

h = Unsupported length of column.

K = Least radius of gyration of a metal pipe section (in pipe columns)

$n = \dfrac{30{,}000}{f'_c}$

p' = Ratio of volume of spiral reinforcement to the volume of the concrete core (out to out of spirals) of a spirally reinforced concrete column.

p_g = Ratio of the effective cross-sectional area of vertical reinforcement to the gross area A_g (See Sec. 1103).

P = Total allowable axial load on a column whose length does not exceed 10 times its least cross-sectional dimension.

P' = Total allowable axial load on a long column.

R = Least radius of gyration of a section; ratio of gross area to core area of a spirally-reinforced concrete column, A_g/A_c.

t = Over-all depth of section (see Sec. 1109).

Section 1101 (*a*)—Change the third sentence to read:

Principal columns in buildings shall have a minimum diameter of *12 in., or in the case of rectangular columns, a minimum* thickness of 10 inches and a minimum gross area of 120 sq. in.

Section 1103 (*a*)—Change formula (22) to read:

$$P = A_g (0.225 f'_c + f_s p_g) \quad (22)$$

**Section 1103* (*b*)—Change last sentence to read:

These spacing rules also apply to adjacent *pairs of* bars at a lapped splice; *each pair of lapped bars forming a splice may be in contact, but the minimum clear spacing between one splice and the adjacent splice should be that specified for adjacent single bars.*

**Section 1103* (*c*) (1)—Change first sentence to read:

For deformed bars—with concrete having a strength of 3000 lb. per sq. in. or above, 24 diameters of bar of intermediate grade steel and 30 diameters of bar of *hard grade* steel.

**Section 1103* (*e*)—Change last line to read:

.....and weathering provisions of Section *507* (*b*)

Section 1103 (*f*)—Delete 3rd and 4th sentences in this Section.

New Section 1103 (*g*)—*Equivalent Circular Columns*

As an exception to the general procedure of utilizing the full gross area of the column section, it shall be permissible to design a circular column and to build it with a square, octagonal, or other shaped section of the same least lateral dimension. In such case, the allowable load, the gross area considered, and the required percentages of reinforcement shall be taken as those of the circular column.

Section 1104 (*a*)—Change the first sentence to read:

The maximum permissible axial load on columns reinforced with longitudinal bars and separate lateral ties shall be *80* per cent of that given by Formula (22).

Section 1105 (*a*)—Change formula (24) as follows:

$$P = 0.225 A_c f'_c + f_s A_s + f_r A_r \quad (24)$$

Section 1106 (*b*)—Change formula (26) as follows:

$$P = 0.225 A_c f'_c + A_r f'_r \quad (26)$$

**Section 1106* (*b*)—Immediately below formula (27), change:

Wherein f'_r = *permissible* unit stress in metal core.

**Section 1108* (*a*)—Change 3rd sentence to read:

In building frames, particular attention shall be given to *effect* of unbalanced floor loads, etc.

*In last line of Section, change:

"their relative stiffness" to "their relative *stiffnesses.*"

Section 1110—Change title to read:

Permissible Combined *Axial and Bending* Stress.

Change paragraph (*a*) to read:

For spiral and tied columns, eccentrically loaded or otherwise subjected to combined axial compression and flexural stress, the maximum permissible compressive fiber stress, f_c, is given by formula (30)

$$f_c = f_a \left[\frac{1 + \frac{ec}{R^2}}{1 + C\frac{ec}{R^2}} \right] \quad \text{.....(30)}$$

wherein the notation is that of Sec. 1103 and 1109 and, in addition, f_a is the average permissible stress on an equivalent axially loaded concrete column, and C is the ratio of f_a to the permissible fiber stress for members in flexure. Thus

$$f_a = \frac{0.225f'_c + f_s p_g}{1 + (n-1)\,p_g} \text{ for spiral columns,}$$

and 0.8 of this value for tied columns. In general $C = \frac{f_a}{0.4f'_c}$

CHAPTER 12—FOOTINGS

New Section 1200—Notations

A = Total area of top of pedestal, pier, or footing at the column base.
A' = Loaded area of pedestal, pier or footing at the column base.
f'_c = Ultimate compression strength of concrete *usually* at age of 28 days.
d = Depth from compression face of footing to center of longitudinal tensile reinforcement.
r_a = Permissible unit working stress in concrete over the loaded area of a pedestal, pier or footing.

For Sections 1201 to 1206 inclusive in the present Regulations substitute new Sections 1201 to 1209 inclusive, as follows:

1201—Scope

(*a*) The requirements prescribed in Sec. 1202 to 1209 apply only to isolated footings.*

1202—Loads and Reactions

(*a*) Footings shall be proportioned to sustain the applied loads and induced reactions without exceeding the allowable stresses as prescribed in Sec. 305 and 306, and as further provided in Sec. 1205, 1206 and 1207.

(*b*) Computations for moments and shears shall be based upon an upward reaction assumed to be uniformly distributed per unit area or per pile unless the footing is eccentrically loaded or the column transmits a moment to the footing, in which cases proper allowance shall be made for any variation that may exist in the intensity of upward reaction consistent with the magnitude of the applied load and the amount of its actual or virtual eccentricity.

1203—Sloped or Stepped Footings

(*a*) In sloped or stepped footings, the angle of slope or depth and location of steps shall be such that the allowable stresses are not exceeded at any section.

(*b*) In sloped or stepped footings, the effective cross-section in compression shall be limited by the area above the neutral plane.

(*c*) Sloped or stepped footings shall be cast as a unit.

*The committee is not prepared at this time to make recommendations for combined footings—those supporting more than one column or wall.

1204—Bending Moment

(*a*) The external moment on any section shall be determined by passing, through the section, a vertical plane which extends completely across the footing and computing the moment of the forces acting over the entire area of the footing on one side of said plane.

(*b*) The greatest bending moment to be used in the design of an isolated footing shall be the moment computed in the manner prescribed in Sec. 1204 (*a*) at sections located as follows:

(1) At the face of the column, pedestal or wall, for footings supporting a concrete column, pedestal or wall.

(2) Halfway between the middle and the edge of the wall, for footings under masonry walls.

(3) Halfway between the face of the column or pedestal and the edge of the metallic base, for footings under metallic bases.

(*c*) The width resisting compression at any section shall be assumed as the entire width of the top of the footing at the section under consideration.

(*d*) In one-way reinforced footings, the total tensile reinforcement at any section shall provide a moment of resistance at least equal to the moment computed in the manner prescribed in Sec. 1204 (*a*); and the reinforcement thus determined shall be distributed uniformly across the full width of the section.

(*e*) In two-way reinforced footings, the total tensile reinforcement at any section shall provide a moment of resistance at least equal to eighty-five per cent (85%) of the moment computed in the manner prescribed in Sec. 1204 (*a*); and the total reinforcement thus determined shall be distributed across the corresponding resisting section in the manner prescribed for square footings in Sec. 1204 (*f*), and for rectangular footings in Sec. 1204 (*g*).

(*f*) In two-way square footings, the reinforcement extending in each direction shall be distributed uniformly across the full-width of the footing.

(*g*) In two-way rectangular footings, the reinforcement in the long direction shall be distributed uniformly across the full width of the footing. In the case of the reinforcement in the short direction, that portion determined by formula (31) shall be uniformly distributed across a band-width (*B*) centered with respect to the center line of the column or pedestal and having a width equal to the length of the short side of the footing. The remainder of the reinforcement shall be uniformly distributed in the outer portions of the footing.

$$\frac{\text{Reinforcement in band-width } (B)}{\text{Total reinforcement in short direction}} = \frac{2}{(S+1)} \qquad (31)$$

In formula (31), "*S*" is the ratio of the long side to the short side of the footing.

1205—Shear and Bond

(*a*) The critical section for shear to be used as a measure of diagonal tension shall be assumed as a vertical section obtained by passing a series of vertical planes through the footing, each of which is parallel to a corresponding face of the column, pedestal, or wall and located a distance therefrom equal to the depth (*d*) for footings on soil, and one-half the depth (*d*) for footings on piles.

(b) Each face of the critical section as defined in Sec. 1205 (*a*) shall be considered as resisting an external shear equal to the load on an area bounded by said face of the critical section for shear, two diagonal lines drawn from the column or pedestal corners and making 45° angles with the principal axes of the footing, and that portion

of the corresponding edge or edges of the footing intercepted between the two diagonals.

(*c*) In the case of footings on piles, the reaction from any pile wholly within the area bounded by the critical sections as defined in Sec. 1205 (*a*) shall be excluded in computing shear.

(*d*) For allowable shearing stresses, see Sec. 305 and 808.

(*e*) The critical sections for bond shall be assumed at the face of the column, pedestal, or wall; at vertical planes where changes in section occur; and near the end of the bending moment reinforcement.

(*f*) For allowable bond stresses see Sec. 305 and Sec. 901 to 905.

1206—Transfer of Stress at Base of Column

(*a*) The compressive stress in the longitudinal reinforcement at the base of a column shall be transferred to the pedestal or footing by dowels. There shall be at least one dowel for each column bar, and the total sectional area of the dowels shall not be less than the sectional area of the longitudinal reinforcement in the columns. The dowels shall extend up into the column and down into the pedestal or footing the distance required for lap of longitudinal column bars (see Sec. 1103).

(*b*) The allowable compressive unit stress on top of the pedestal or footing directly under the column shall not be greater than that determined by formula (32).

$$r_a = 0.25f'_c \sqrt[3]{\frac{A}{A'}} \qquad (32)$$

(*c*) In sloped or stepped footings, A may be taken as the area of the top horizontal surface of the footing, or as the area of the lower base of the largest frustum of a pyramid or cone contained wholly within the footing and having for its upper base the loaded area A', and having side slopes of one horizontal to two vertical.

1207—Pedestals and Footings (Plain Concrete)

(*a*) The allowable compressive unit stress on the gross area of a concentrically loaded pedestal shall not exceed $0.25f'_c$. Where this stress is exceeded, reinforcement shall be provided and the member designed as a reinforced concrete column.

(*b*) The depth and width of a pedestal or footing of plain concrete shall be such that the tension in the concrete shall not exceed $.03f'_c$, and the average shearing stress shall not exceed $.02f'_c$ taken on sections as prescribed in Sec. 1204 and 1205 for reinforced concrete footings.

1208—Footings Supporting Round Columns

(*a*) In computing the stresses in footings which support a round or octagonal concrete column or pedestal, the "face" of the column or pedestal shall be taken as the side of a square having an area equal to the area enclosed within the perimeter of the column or pedestal.

1209—Minimum Edge-Thickness

(*a*) In reinforced concrete footings, the thickness above the reinforcement at the edge shall be not less than 6 in. for footings on soil, nor less than 12 in. for footings on piles.

(*b*) In plain concrete footings, the thickness at the edge shall be not less than 8 in. for footings on soil, nor less than 14 in. above the tops of the piles for footings on piles.

The report of Committee 501 will go before the 36th Annual Convention, Feb. 27-29, 1940, on motion for adoption of the proposed revisions of the 1936 Tentative Code. Discussion is scheduled for the June 1940 JOURNAL and (in triplicate) should reach A. C. I. Secretary by Apr. 1, 1940.

JOURNAL
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AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN JANUARY 1940

Errata—Building Regulations for Reinforced Concrete (A. C. I. 501-36T)*

American Concrete Institute Committee 501, Standard Building Code, supplies the following list of corrections of editorial and typographical errors which appear in "Building Regulations for Reinforced Concrete" (A. C. I. 501-36T) of 1936.†

Page No. *Section No.*

3 102(a). In 2nd line, change "sizes" to "size."

3 103(a). In 2nd and 3rd lines, change to read: "and the design of which is either in conflict with, or not covered by these regulations."

4 104(a). Change the following definitions to read as follows:

4 *Composite Column:* A column in which a steel or cast-iron section is completely encased in concrete containing spiral and longitudinal reinforcement.

4 *Deformed Bar:* Reinforcing bars with closely spaced shoulders, lugs or projections formed integrally with the bar during rolling. Wire mesh with welded intersections not farther apart than twelve inches in the direction of the principal reinforcement and with cross-wires not smaller than No. 10 may be rated as a deformed bar.

4 *Diagonal Band:* A group of reinforcing bars covering a width approximately 0.4 the average span, placed symmetrically with respect to the diagonal running from corner to corner of the panel of a flat slab.

4 *Direct Band:* A group of reinforcing bars, covering a width approximately $0.4l_1$, placed symmetrically with respect to the center lines of the supporting columns of a flat slab.

4 *Drop Panel:* The structural portion of a flat slab which is thickened in the area surrounding the column capital.

4 *Effective Area of Concrete:* The area of a section which lies between the centroid of the tensile reinforcement and the compression face of the structural member.

4 *Effective Area of Reinforcement:* The area obtained by multiplying the right cross-sectional area of the reinforcement by the cosine of the angle between its direction and the direction for which the effectiveness is to be determined.

*JOURNAL, American Concrete Institute, March-April, 1936; *Proceedings*, Vol. 32, p. 407 (and in separate pamphlets).

†Page references are to separate pamphlet publication. For the original JOURNAL publication and in the annual Volume (32), 1936, the Section numbers will provide a basis for corrections.

4 *Plain Concrete:* Concrete without reinforcement, or reinforced only for shrinkage and temperature changes.

4 *Ratio of Reinforcement:* The ratio of the effective area of the reinforcement to the effective area of the concrete at any section of a structural member.

5 *Reinforced Concrete:* Concrete in which the reinforcement, other than that provided for shrinkage or temperature changes is embedded in such a manner that the two materials act together in resisting forces.

5 104(b). Change notation to read as follows:
A_v = Total area of web reinforcement in tension within a distance of s (measured in a direction parallel to that of the main reinforcement), or the total area of all bars bent up in any one plane.

8 204(a). In 2nd line change C9-30 to C9-38; also "Tentative Specifications for High-Early Strength Portland Cement" to "Standard Specifications for High-Early Strength Portland Cement."
In 3rd line, change C74-30T to C74-39.

8 205(a). In 2nd line change C33-31T to C33-39.

8 207(a). At the close of the section add: "or for "Axle-Steel Concrete Reinforcement Bars" (A. S. T. M. Serial Designation: A160-36)"
207(b). At the close of the section add: "Or for "Welded-Steel Wire Fabric for Concrete Reinforcement" (A. S. T. M. Serial Designation: A185-37)."

8 207(c). In the second line change A9-34 to A9-36.

9 208(a). In the 1st line omit "at the work."

9 301(b). Change footnote to read: *In climates where frost action is not severe this section should be omitted.

10 303(d). In 1st line omit "In all cases".
In 2nd line omit "shown by these tests."
In 4th line change "in the mix" to "in the mixture."

10 304(b). In 3rd line change "Tentative Specifications" to "Standard Specifications."
In 4th line change C94-35 to C94-38.

11 305(a). Change 2nd and 3rd lines to read: "where f'_c equals the minimum specified ultimate compressive strength at 28 days."
In Table of Allowable Unit Stresses, move parenthetical statement "(where special anchorage is provided (see Sec. 903a and 905a) one and one-half times these values in bond may be used)" to a position immediately following "In beams and slabs and one way footings," since this provision does not apply to two-way footings.

12 401(a). In 2nd line change "from the places" to "from the spaces."
402(c). In 2nd line change "Tentative Specifications" to "Standard Specifications."
In 3rd line change C94-35 to C94-38.

14 505(a). In 5th line change "the minimum clear spacing" to "the minimum clear distance."
507(a). Revise the 2nd sentence to read: "If concrete surfaces after removal of the forms are to be exposed to the weather or be in contact with the ground, the reinforcement shall be protected with not less than 2 inches of concrete."
507(b). Revise the 1st sentence to read: "The concrete protective covering for reinforcement at surfaces not exposed directly to the ground or

weather shall be not less than ¾ inches for slabs and walls; and not less than 1½ inches for beams, girders and columns."

15 601(a) (2). In 2nd line change "shall be taken" to "shall be assumed."

16 603(c). In 3rd line change "provided in Sections 305 and 306" to "provided in sections 305, 306 and 1110."

17 702(b) (4). Change 3rd line to read: "For structural, intermediate and hard grades, and rail and axle steel.....0025 *bd*."

18 706. Change heading to "Concrete Joist Floor Construction."

18 706(a). Change section to read: "Concrete joist floor construction consists of concrete joists and slabs placed monolithically with or without burned clay or concrete tile fillers. The joists shall not be farther apart than 30 inches face to face. The joists shall be straight, not less than 4 inches wide, nor of a depth more than 3 times the width."

18 706(b). In 2nd line change "in the ribs" to "in the joists."
In 4th line change "with the ribs" to "with the joists."

18 706(c). In 2nd line change "clear distance between ribs" to "clear distance between joists."

18 706(d). In 3rd line change "ribs" to "joists."
In 4th line change "to the ribs" to "to the joists."

19 709(a). In 2nd, 3rd and 4th lines change "ribs" to "joists."

21 709(h) (2). In 2nd line change "formula (12)" to "formula (10)."

22 710(a). In 1st line change "ribbed floor construction" to "concrete joist construction."

22 801(a). Revise 1st and 2nd lines to read: "The shearing unit stress(v), as a measure of diagonal tension, in reinforced concrete beams shall be computed by formula (14):"
801(c). In 1st line change "In ribbed construction" to "in concrete joist floor construction."
In 4th line change "Sec. 707(b)" to "Sec. 706(b)."

23 807(a) (1). Revise to read: (1) $0.03\,f'_c$, when at least 50 per cent of the total negative reinforcement in the column strip passes directly over the column capital."

(2) $0.25f'_c$, when 25 per cent of the total negative reinforcement in the column strip passes directly over the column capital."

33 1103(b). Change last sentence to read: "These spacing rules also apply to adjacent pairs of bars at a lapped splice; each pair of lapped bars forming a splice may be in contact, but the minimum clear spacing between one splice and the adjacent splice should be that specified for adjacent single bars."

33 1103(c) (1). Change 2nd and 3rd lines to read: "30 diameters of bar of hard grade or rail steel."

34 1103(e). Change last line to read: "and weathering provisions of Section 507(b)."

36 1106(b). Immediately below formula (27) change to read: "wherein f'_r = permissible unit stress in metal core."

37 1108(a). Change 3rd sentence to read: "In building frames, particular attention shall be given to effect of unbalanced floor loads, etc."

37 1108(a). In last line change: "their relative stiffness" to "their relative stiffnesses."

40 1205(c). Change 4th line to read: "and having side slopes of one horizontal to two vertical."

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Discussion of Report of Committee 501, Standard Building Code:

Proposed Revisions of "Building Regulations for Reinforced Concrete"*

(A. C. I. 501-36T)

BY F. R. MCMILLAN, A. W. STEPHENS, M. O. WITHEY, T. D. MYLREA, JOHN R. NICHOLS, W. A. COOLIDGE, F. E. RICHART, JOSEPH DISTASIO, DEAN PEABODY, JR., C. A. WILLSON, FREDERICK G. ANDERSON, WALTER H. WHEELER, JAMES A. MCCARTHY, NATHAN M. NEWMARK, R. D. BRADBURY

CONVENTION DISCUSSION

F. R. McMillan, presiding officer—We will now consider Proposed Revisions of "Building Regulations for Reinforced Concrete." In 1936 modifications were made in the 1928 code. Institute Committee 501 is submitting proposed revisions to the 1936 tentative document and will, on the approval of those revisions, propose that the revised code be adopted as a standard of the Institute. Each of these revisions can be adopted by majority vote; they have all been approved by the committee itself. They may be amended on the floor of this Convention, only by a nine-tenths vote of the members present and voting. After the revisions on each separate chapter are presented, the revisions of the whole chapter will be placed before you for a vote unless there is a request for a vote on the individual changes. Any one in the room, members or non-members of the Institute, are privileged to participate in discussion but non-members of the Institute will refrain from voting. Mr. A. W. Stephens, Chairman and Prof. R. R. Zipprodt, Secretary of A. C. I. Committee 501, Standard Building Code will present the report.

*JOURNAL, Amer. Concrete Inst., Jan. 1940.

(Note: Chapters and Sections will be recorded here only by reference to the report published in the JOURNAL for January 1940 and to the tentative code of 1936.—EDITOR)

Mr. Stephens—I refer first to Chapter 1—General, definitions and notations. In the proposed revision notations have been removed and placed at the head of each chapter to which the particular notations pertain, reducing the length of this chapter and we believe making our regulations much more usable. In Section 102 (a) we have added, just before the last sentence in that Section, the sentence, "All plans submitted for approval or use on the work shall clearly show the strength of concrete for which all parts of the structure were designed."

In Section 103 (a) we have changed the sentence in the tentative regulations in the second and third lines, to read "and the design of which is either in conflict with, or not covered by these regulations. Otherwise the contents of the chapter, which have been before the Institute for some years, are unchanged except for changes in definitions whereby we hope that they are made a little clearer, and the omission of four definitions which we believe are covered adequately elsewhere in the Code or else are no longer necessary. (This chapter approved without dissent.)

Mr. Stephens—In Chapter 2—Materials and Tests, we have added the notations which refer to the text and changed serial designations to conform to the latest action of the American Society for Testing Materials Committees. Section 206 (a) has been changed slightly in its wording from the tentative standard, "Water used in mixing concrete shall be clean and free from injurious amounts of oils, acids alkalis, organic materials or other deleterious substances. Water normally used for drinking purposes is satisfactory." (Attention was directed to other minor changes in Chapter 2 and the chapter was approved without dissent.)

In the presentation of Chapter 3—Concrete Quality and Working Stresses, question was raised under the heading "Bond of Plain Bars."

*M. O. Withey**—What is the ultimate bond strength of one inch plain round bars with a concrete strength of around 3500 to 4000 p.s.i.

T. D. Mylrea† (member Com. 501, called upon by Mr. Stephens)—About 260 to 300 p.s.i.

Professor Withey—150 p.s.i. is too high for a safe working stress for bond on plain round bars in concrete having a 3750 p.s.i. ultimate. My tests would not support 150 p.s.i. on round one-inch bars with a

*Professor of Mechanics, University of Wisconsin.
†Head, Division of Civil Engineering, University of Delaware.

factor of safety of two. I wondered whether the committee had considered these values in the light of tests made by the Portland Cement Association about a year ago and tests by Professor Gilkey which have also been reported here. If this has all been considered carefully, in the light of all the information we have, probably they are safe, but they look very high to me.

Professor Mylrea—The values proposed here are at least two years old; more recent ones have not been taken into account because no recent committee action has been taken. I believe, however, that the values given are safe.

(The matter in question was referred back to the Committee for report the following day. Mr. Stephens proceeded with the presentation of Chapter 3.)

*John R. Nichols**—As I see it, there is no limitation on the span of one-way slabs with working stresses of 30,000 p.s.i. in the steel, and consequently no limitation on deflections that may occur?

Professor Mylrea (again called on by Mr. Stephens)—There are a number of points concerned in that 30,000 lb. maximum, some of which Mr. Stephens has referred to. First, as the result of a large number of tests, we found that the 30,000 lb. specification was easily permissible. Tests were made on 8-ft. spans and continuous spans of 16 ft. We made tests of bars up to $\frac{5}{8}$-in. diameter. The tests on $\frac{5}{8}$-in. bars were not altogether satisfactory, but limiting the size to $\frac{1}{4}$-in. bars practically ruled the bars out altogether. Members of the Concrete Reinforcing Steel Institute objected on the ground that that permitted wire to be used but not bars, and we changed that to three-eighths bars, which we thought was in accordance with the tests made. With the $\frac{3}{8}$-in. tension bars on long spans, bars will be so crowded that construction will not be feasible. If you attempt to use bars larger than three-eighths, that also will not be feasible. Although not so explicitly stated, it seems to me the specification will work automatically.

Mr. Nichols—May I ask further on this very same point: The tests made, I understand, were on 8-ft. spans with bars up to $\frac{5}{8}$-in., which these tests proved satisfactory. If $\frac{5}{8}$-in. bars had been used on 12-ft. spans and suitable depth of slab for that length, isn't it likely that such bars would have proved as satisfactory as three $\frac{3}{8}$-in. bars with 8-ft. spans?

Professor Mylrea—I think not.

*Consulting Engineer, Boston.

(The discussion was resumed the following morning with Prof. F. E. Richart in the Chair.)

Mr. Stephens—I do not see Professor Mylrea here this morning, but it is my belief that he would agree that we failed to take into consideration some tests that have been made available in the last year or two, indicating that stresses allowed in Chapter 3 for concrete of high strength may be a little too high. This leads to a problem, one of several, perhaps, in connection with our Code. There is a question as to what extent the Code can be changed by the Convention, although I believe that changes must have the approval of nine-tenths of those present and voting. I would like to have your consent to proceed with other sections of the Code, leaving the question of voting on the individual chapters to be taken up after we have gone through the Code. In certain chapters of the Code the changes proposed are very important. Requirements differ radically from some of the requirements to which we have been accustomed heretofore, and we hope to have a very free and thorough discussion, and it is not at all unlikely that out of that discussion will grow the necessity for making certain additional changes.

The committee, in studying certain of the chapters, notably ten, eleven and twelve, has given a great deal of thought to them, has worked intensively, and the results of that study have been before you since the publication of the January JOURNAL, not a very long time. Since the committee has been in attendance at the convention, there has grown into its consciousness a thought that there is a question, a very serious question, whether or not the engineering profession at large which we represent, have had, or will have had, sufficient opportunity for thorough study of the proposed code requirements. The thought has come to us that perhaps you may decide that more time should elapse before we ask the Code's acceptance. That is for your determination in the Convention this morning.

With that thought in mind, I should like to present the balance of the Code, so that we may have the full benefit of your thought and discussion.

(Mr. Stephens presented Chapter 4—Mixing and Placing Concrete, and there being no discussion or objection proceeded to Chapter 5—Forms and Details of Construction.)

Mr. Stephens—Section 502 (a). At the end of Section 502 (a) we add the following: "The results of suitable control tests may be used as evidence that the concrete has attained sufficient strength." In order to understand the meaning of the sentence, it is necessary to

refer to the text. (Chapter 6, Design—General Considerations, was presented.)

*W. A. Coolidge**—I find the old Code says that the ultimate compressive strength of concrete is usually at age of 28 days. I have had trouble with that specification. I think it would be safer to say 28-day concrete, unless otherwise specified. That term usually can be used. The definition of f_c, does not specifically state what the 28-day strength of concrete should be. I think we should specifically state "Cured 70°F. moist and broken wet." That specification too, has led to trouble in several cases, and we all realize that if we dry a cylinder out at the end of three weeks, we can gain about 25 per cent in strength.

Mr. Stephens—I believe that the twenty-eight day compressive strength is related to the A.S.T.M. definition in the matter of control cylinders, and that the application should be clarified. I wonder if any of our committee has anything to say about revising this particular chapter in reference to the point made by Professor Coolidge?

Professor Richart—I happen to be familiar with this Section in the old Code and in the 1936 revision of the Code, and we have a very specific statement of the way these tests shall be made, (Sec. 303 (c): "The standard age at test shall be 28 days, but 7-day tests may be used provided that the relation between the 7- and 28-day strengths of the concrete is established by tests for the materials and proportions used." There can be no doubt as to just what is meant. I think we should take under advisement the wording you suggest replacing the word "usually" by the words "unless otherwise designated."

Professor Coolidge—You have not answered the question with respect to where, in the old specifications, there is a statement as to how the cylinder should be broken, whether wet or dry, which makes considerable difference.

Professor Richart—I quoted one of four paragraphs of Sec. 303, the first of which, as Mr. Stephens says, refers to the method of making tests. That is definitely included in the 1936 Code.

(Mr. Stephens then proceeded to present Chapter 7—Flexural Computations, and in connection with the presentation, made the following statement:)

Mr. Stephens—Following what has been our practice when a material has received acceptance by the A.S.T.M., we proceed to give consideration to its use in connection with our Code. A change in Section 702 (b)(4) was necessitated by the fact that axle steel provided for

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under the requirements of the American Society for Testing Materials and accepted by that organization for use as reinforcement, has been included in our Code as permissible, and that required a change. There have been certain changes in the numbering of the sections which you will note. In order to be consistent throughout our Code, we are using the word "joist" in connection with a certain type of construction instead of the word "ribbed." In reference to Section 707 (a) "Change Section to read: 'Concrete joist floor construction consists of concrete joists and slabs placed monolithically with or without burned clay or concrete tile fillers. The joists shall not be farther apart than 30 in. face to face. The joists shall be straight, not less than 4 in. wide nor of a depth more than three times the width.' " That simply changes the word ribs to joists.

In Section 709 (a) Remove completely the last part of the first sentence as follows: "And concrete ribs with top slabs placed monolithically with the ribs." A question has been asked concerning that change and we have been unable, with the information available and from the records of our committee meetings, to determine just how and when that change was made, and I will right now ask some member of that committee if he has any recollection of why that change was made and what it was about. Mr. DiStasio can you answer that question?

*Joseph DiStasio**—I recall that the question came up whether the tile fillers were to be taken in moment or shear, and there was considerable discussion on that and the committee did not come to a definite conclusion on it so it was eliminated. That is my recollection of it. I might say this, that, in New York the same question came up a few weeks ago, and we ruled that in two-way slabs, with a concrete rib in contact with and surrounding a block on all sides, that the tile filler, both the web and the flange would be taken into consideration in the flexural computations, and that in one-way slabs where the joints of the fillers were staggered, that there be allowed only half value of the web and the flange; and I do not believe that this committee has yet made definite recommendations on the tile filler as far as the American Concrete Institute is concerned. That is my recollection of the matter.

Mr. Stephens—This question did arise a year or two ago; it involves a distinct type of construction used quite generally in certain sections of the country. We have a letter from one of our engineers who has used that system and he raises the question concerning the reason for that removal because it very definitely affects his business. It goes

*Consulting Engineer, New York.

without saying that our decisions on engineering problems are not, of necessity, greatly influenced by a man's business; on the other hand, we need to be very sure that every decision is based upon some sound engineering principles and that thorough consideration has been given to all the facts. I should like to have an expression of opinion from members present concerning their thought in the matter. (No response.) The amount of shear transference value in two-way slabs, when the construction consists of ribs running in both directions separated by 16, 18 or 20 in. and having on top a continuous slab, (possibly a 2-in. slab) is to be compared with that of a solid slab in which the amount of shear transference is considerable. Comparison may also be made with those types of construction in which spaces between the concrete ribs are filled with clay tile or other types of fillers, in which the construction is such that and in which there is a considerable amount of shear transference value contributed by the fillers. It will, I think, help the committee to receive comments or suggestions.

Mr. Nichols—I do not think the question of shear transference or the value of the shear enters into the question of striking out this phrase. The sentence in which that deletion occurs tells what kind of slabs are slabs supported on four sides. As the sentence was in the 1936 Tentative Code, such slabs should be solid reinforced concrete slabs, concrete ribs or joists, with burnt clay or concrete tile fillers, with or without top slabs and concrete slabs with top slabs but without any fillers. Now, if this phrase is stricken out, apparently you will not allow design under this section of a slab which has open cells on the under side, what we call two-way rib construction without the fillers would not be allowed to be designed under this section. Of course engineers would design under this section if they wanted to, but building codes might forbid it if this were stricken out. There does not seem to be any reason why it should be stricken out, and I don't know why it was done.

*Dean Peabody, Jr.**—I merely wish to add that we may be in an inconsistent position in allowing these slabs without fillers for one-way slabs and flat slabs, but not for two-way slabs.

C. A. Willson†—It says in Sec. 709 (c) "When a concrete top slab, placed monolithically with the ribs is used, it shall be not less in thickness than one and a half inches nor less than one-tweltfh of the clear distance between ribs." The proposal is to omit 709 (c). I would assume that if we omit 709 (c), we could use one inch of concrete

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on top of the clay tile fillers. I suppose the purpose of omitting 709 (c) would be to permit the use of clay tile fillers with concrete ribs, but with no topping at all, and I approve of that. I think if some concrete topping is used over the clay tile fillers, I would prefer to see some minimum thickness specified, one and a half inches, or, according to my own practice, two or even two and a half inches preferred, but if we omit any use of 709 (c), we can allow anything we want, we can allow one inch, we can allow three-quarters of an inch. I doubt if we intend that interpretation.

Mr. DiStasio (to whom Mr. Stephens referred the question)—I think Mr. Willson is correct, because in the committee there was much discussion of what to allow in the computation for fillers. It is my recollection that we came to no definite agreement so that some one suggested the entire section be deleted. Now, if that deletion has caused another change, then of course it should be corrected, but I agree with Mr. Willson that if we use the topping, it should be at least an inch and a half. By the Shuster or the Republic systems without any concrete topping it is all right, but it was the practice in the old days to call for a concrete slab where they needed it. I have seen as much as a quarter or a half inch of concrete, considered as adding to the strength—the tile was an inch and by adding half an inch of concrete it was considered an inch and a half. I believe that an inch and a half is the proper minimum thickness to add. The New York Code requires an inch and a half of concrete topping.

Mr. Nichols—709 (c) in the Tentative Code is a clause which governs tile and joist and it ought to be called a tile and joist construction and has no particular bearing on slabs supported on four sides. The section which used to be 706 (c), contains a paragraph which covers precisely the same thing under the heading which used to be one-way ribbed construction. The heading has been changed to concrete joist construction, and as such it covers concrete joist construction, whether one-way or two ways. There is the right place for specifying what concrete joist construction shall be like, whether one-way or two-ways, and that paragraph now governs the matter for both types of construction and the paragraph is unnecessary in Section 709 and was therefore left out. (In response to request Mr. Stephens read Sec. 709, paragraphs b, c and d.)

Mr. Nichols—I think, just to make this complete, that those three paragraphs all go together; they are covered in Section 706 (b) (c) and (d) under a heading which used to be "One-way ribbed floor construction" but has now been changed to "Concrete joist floor construction" and therefore now applies both to one-way and two-way con-

struction, and there is no need to repeat those paragraphs which govern the tile construction.

Mr. Willson—I think Mr. Nichols is correct and I withdraw my statement.

Chairman Richart—Is there any further discussion on this Section? The Secretary will note the question that has arisen concerning this omission embodied in the words "and concrete ribs with top slabs placed monolithically with the ribs" which was included originally under Section 709 (a). We will return to that later.

Mr. Stephens—We have a new section 709 (b).

Mr. Nichols—May I make a suggestion in regard to 709 (a), the long sentence, which reads as follows: "709 (a) This construction, consisting of floors reinforced in two directions and supported on four sides, includes solid reinforced concrete slabs, concrete ribs with burned clay or concrete tile fillers, with or without concrete top slabs; and concrete ribs with top slabs placed monolithically with the ribs. The supports for the floor slab may be walls, reinforced concrete beams or steel beams fully encased in concrete."

Now when it comes to walls, I suppose a wall may be any kind of a wall that is capable of supporting the end of the slab. A masonry wall, for example, is a wall and is not ruled out by this phrase. Suppose the edge of the slab is supported by a steel beam which is not encased in concrete, what is the matter with the slab in that case? Why is it not perfectly good construction? Why must it be encased in concrete, if all you want to do is to support the end of the slab?

Mr. DiStasio (to whom the question was referred)—I think that since some edge restraint is required, the intended distinction between wall-bearing and an incased steel beam, is that with a wall bearing there would be effective support when the slab is built into the shell of the wall, while with the encased steel beam there would be no negative support. I am not sure we could not amend that to improve the wording, because I can visualize that a parapet wall would give the same result as an encased steel beam. In a two-way slab you do get some restraint at the end, no matter how small.

Mr. Nichols—I would like to offer a suggestion, if this is the proper place to offer the suggestion to the committee, that such slabs may be supported upon walls or beams and it does not make any difference what kind of walls or beams they are; and if you do not specify what kind of walls or beams they are, you strike out a lot of words. If it is necessary in connection with slabs supported on four sides that the corners be held down, and that, I understand, is the real essential for this

type of design, then require that such slabs have the corners held down. The corner of a slab can be held down by a steel beam without encasing it in concrete.

Mr. DiStasio—I have no objection to that suggestion at all. I am just wondering how practical it is to anchor the corners down; you would probably have to have anchor bolts at the corners and plates to hold them down. I would like to think about it. We might allow the support on beams and walls without any qualifications. I am more or less inclined to agree with Mr. Nichols.

*Frederick G. Anderson**—If using a beam with an encasement of concrete does not alter the strength of the joist, I do not see the need for calling for encasement; it will add strength, but in some cases strength not needed.

Mr. DiStasio—I think Mr. Nichols means the general subject of two-way slabs and he is not referring just to joists but also to solid slabs and whether or not to hold the corners. Now we do have cases where the support is simply a reaction and becomes similar to a one-way slab. I am inclined to agree with Mr. Nichols on allowing that revision.

Walter H. Wheeler†—It seems to me there is absolutely no reason why the two-way slab cannot either rest on walls or beams or have the beams enclosed or have restraint at the beams, but under the condition of resting on the wall or beam, the basis of design would have to be modified to take that into consideration.

Mr. DiStasio—That is where we call for a factor, one which really is the same as for a one-way slab in the Code.

Professor Coolidge—Couldn't we clarify this whole thing a great deal if you made, in the beginning, a definition of what is meant by restraint, the minimum restraint, in the solid slab specification? If defined here, it would simplify the whole design, because it affects the choice of the factor "one" or what not.

Mr. DiStasio—Where we lack restraint, we have made the factor $e_a = 1.00$, which really takes into consideration no restraint at corners. For practical reasons we still like to see some restraint. I guess that was the reason we differentiated between the wall bearing and the encased concrete beam.

Mr. Wheeler—It would simplify the whole thing if that were specified in the beginning.

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†Designing and Consulting Engineer, Minneapolis.

Mr. Stephens—Well, if we take the suggestion of Mr. Nichols, I don't think that is necessary, because the unity factor will take that into consideration.

*James A. McCarthy**—Referring back to Section 702, Span Lengths, are there not some possible cases where we would have different span lengths for the slabs? For the encased beams we would have a span length of clear distance between centers of beams; clear span and depth of the beam or slab, when you take the encasement away, will increase; but when we add to that the depth of the beam or slab, that is nullified because we have a narrow flange, so that we shall not need the distance between centers of the supports.

Mr. Stephens—Will Mr. Nichols, who precipitated this discussion, answer that point?

Mr. Nichols—It seems to me that matter will take care of itself. If you wish to support a two-way slab upon encased steel beams, then of course you state what that span length should be. If you wish to encase these beams, the span length would be different. It is all provided, and I think consistently provided, and I assume that the method provided here for designing a two-way slab shows how to design a two-way slab that is freely supported but with no restraint whatever except that the corners are held down. I assumed that method would be applicable if you have such a slab resting on a masonry wall or encased steel beams or any other kind of support that gives you no appreciable support and restraint.

Mr. Stephens—Is there any further discussion? If there is not, I would like to proceed with the next section which we are presenting; that is, new section 709 (b) "Minimum Slab Thickness." (Mr. Stephens then proceeded with the presentation of Sec. 709 (b) and 709 (c), and added:)

We have made no change in the tensile requirements which have been in effect since 1936. We have added in a footnote, largely for the benefit of the designing engineer, certain coefficients and tabulations which we believe will help him in the utilization of this particular chapter of the Code. It has been found that some engineers were having difficulty in the use of this chapter, and these footnotes are added to remove those difficulties.

Section 709 (d) embodies no change in requirements, so we will pass it.

Nathan M. Newmark†—Why is there an apparent inconsistency in the minimum ratio of reinforcement in Sections 4 and 5 of 709 (d)?

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Professor Richart—I will read these two clauses:

"(4) The spacing of the reinforcement shall be not more than 3 times the slab thickness and the ratio of reinforcement shall be at least 0.0025.

"(5) The amount of reinforcement in any one foot wide strip shall be at least 0.003 times the product of the width of strip by the effective depth."

I think we would understand that that was the ratio of reinforcement there, so we have 0.0025 in one case, and 0.003 for any one-foot wide strip in the other case. I believe the point is that it is hard to see how there could be that difference.

Dr. Newmark—It appears to be the intent of these regulations that the minimum ratio of reinforcement shall be three-tenths of one per-cent, regardless of where you take the strips.

Mr. DiStasio—A reduction of 25 per cent was made on the outer quarter.

Dr. Newmark—It does not appear to be stated here.

Mr. DiStasio—I would really like to reconcile in my own mind where this came from, because I am not familiar with where the twenty-five hundredths and three-tenths came from unless a change was made later. Originally it was twenty-five hundredths; I don't know how that change got into the text.

Professor Richart—If the value for any one-foot strip were smaller than the other, we could understand it, but to say it shall be more than the other is a little difficult to understand.

Mr. DiStasio—Except that you are permitted to make a reduction of 25 per cent and take the middle half, but I do not believe that is necessary.

Professor Richart—It seems to me that any one-foot strip would include the outer quarter.

Mr. McCarthy—Why is it necessary to compare two depths? In the first place, isn't the coefficient .0025? Why is it not comparable to the minimum temperature requirements? Why not take the total thickness of the slab and not use the effective depth? I think that calls for information.

Mr. DiStasio—I would not have any objection to using .0025 or changing to .00225, because there is very little difference in it, but if I understand correctly, the difference between the two figures was the question, and it is my opinion that that one-foot strip would cover the middle half. Would you not use the six inches in order to get effective depth for the slab?

Mr. McCarthy—You would have to use the effective depth for that six inch slab, which may be five inches.

Mr. DiStasio—That may be the explanation, but I do not believe that is the intention there.

Mr. Stephens—Is there any further discussion? Several points of importance have been brought out here. Apparently it is necessary to clarify these requirements and give some thought to the relationship of the requirements in this section with the method used in the first part of the chapter concerning temperature reinforcement. I think we ought to bring them to a consistent relation. Our Secretary will record these various points and they will be given consideration, of course. Is there any further discussion?

(There were no comments on Chapter 8—Shear and Diagonal Tension, and consideration was given to Chapter 9—Bond and Anchorage.

Dr. Newmark—904 (a) refers to the anchorage of single, separate bars used as web reinforcement. 904 (a) (3) previously required embedment in the compression area of the beam, and has been changed to embedment above the mid-depth of the beam a distance sufficient to develop the stress to which the bar will be subjected, etc. Is there any reason why that should not read embedment above or below the mid-depth of the beam?

Mr. Stephens—I am looking for Professor Mylrea. I would like to ask some one better versed than I am to answer that question. I am not sure that I can answer this. It seems to me that the original version was preferable. It is obvious that at the point of negative moment, the bond stress of the bar would be determined below the mid-depth of the beam. I think Dr. Newmark contends that in such a case the bond stress involves the portion of the bar in the lower half of the beam. I think the section was changed for simplification, but I believe that Dr. Newmark's suggestion that the word "above" be replaced by "above or below" is a good suggestion. (Mr. Stephens presented a change in Sec. 905 (a) as follows:)

"All bars in footing slabs shall be anchored by means of standard hooks."

Mr. McCarthy—The question I asked yesterday in connection with Sec. 305 and 903 (a) refers specifically to defining what special anchorage shall be. Now I know from personal experience that there has been some difficulty as to the interpretation of this description of special anchorage in the consideration and design of footings. I don't know why we cannot simplify that description of special anchorage when it does apply to footings and not have the term "in the region

of compression" which does limit some one trying to get a higher bond stress on unanchored bars in one-way footings.

Member—In reference to special anchorage as provided by hooks in footings—the statement regarding hooks being placed in the region of compression is supposed to apply to the question of beams and girders and similar structures. Section 905 is a separate section which applies specifically to footings. It says that all longitudinal reinforcement in footings shall be provided with standard hooks, and there is no mention in that section as to the region of compression or anything else; so I think it is perfectly clear. Even in the old code Sec. 905 has the title "Anchorage of Bars in Footings and Slabs." It clearly states that all bars in footings and slabs except the longitudinal reinforcement between loads in continuous slabs and footings (which incidentally is not treated in this Code) shall be anchored by means of standard hooks; the outer faces of these hooks shall be not less than three inches or more than six inches from the face of the footing. So there is nothing confusing there as to the region of compression.

Mr. McCarthy—You refer to special anchorage, and the only interpretation of special anchorage is at the beginning of 903.

Member—I see your point and we will take that under consideration. It is suggested to the Secretary, in 904 (a) paragraph (3), we find bond stress limited to not more than .04 f_c. There is pending a further limitation upon bond stress with very strong mixtures of concrete, and this last must not be overlooked when that is done.

Mr. Stephens—The Secretary will note that. We have arrived now at Chapter 10—Flat Slabs—Two-Way and Four-Way Systems with Square or Rectangular Panels. It is exceedingly important. I am going to ask Professor Peabody to present this chapter to you.

Professor Peabody—Mr. Stephens asked me to give you a brief resume' of the work done by the Sub-committee on Flat Slabs before submitting these new regulations. When the 1936 regulations were adopted Committee 501 considered that the time had come to remove a footnote to the flat-slab regulations stating that the committee was not ready to make recommendations for unequal spans. While we know very well that flat slabs are designed and built as continuous floor systems with unequal spans, our regulations so far have not covered that case. The chairman of this sub-committee had to undertake the problem of expanding our localized regulations for approximately equal spans and approximately square panels into general regulations for all types of floor systems.

There exist, of course, exact solutions of flat slab floors, notably Dr. Westergaard's, but these are only brought down to a form usable

by a designer for the case of rectangular panels with equal spans in a longitudinal direction and other equal spans transversely. The moment coefficients recommended for such cases were those given by an exact analysis modified by the results of flat slab tests. The modified coefficients enabled the designer to compute the slab as a continuous one-way slab longitudinally, and a continuous one-way slab transversely.

The sub-committee chairman, Mr. Bertin, spent almost two years years on the problem of flat-slab solutions for unequal spans. He employed elastic frame analysis, taking a continuous bent, one bay wide, longitudinally, or transversely, and including the columns above and below the floor. The elastic frame analysis used center line dimensions, considering the slab to be a continuous frame centered on the center line of columns. The cross section and moment of inertia of the slab vary as sections are taken in the slab proper, or through the drop panel; similarly the moment of inertia of the column varies as sections are taken in the column proper or through the capital. Mr. Bertin created an approximate method of solution based on constant moment of inertia for the slab and the column. The elastic frame analysis gave moment diagrams for the continuous frame based on center line dimensions for a width of the bay. When such a diagram is used with beam and girder frames, it is customary in beam design to employ for the maximum negative bending moment the moment at the face of the column, instead of the moment at the center of the column. Similarly, the maximum column moment is taken at the top of the slab or the bottom of the beam above, instead of the greater moment at the center line of the slab. Mr. Bertin decided to use this device of the location of a "critical section" for a reduction of the maximum negative moment instead of an arbitrary percentage reduction as has been employed in the past for the special case of square panels and equal spans. The arbitrary reduction, you will remember, amounts to 28 per cent.

The "critical section" was selected by a study of panels of equal span to give the same negative moments recommended by the Joint Committee. The distance to this "critical section" is given in the new specifications and usually locates a section beyond the column capital, somewhere in the drop panel. The negative moment at this critical section agrees very closely with the Joint Committee values for slabs with drop panels, but is ten to twenty per cent too high for slabs without drop panels. The greater spread, the fifteen to twenty per cent values, occurs for panels supported by columns having small capitals, or no capitals at all. We find that this continuous frame

analysis gives positive moments which are very close to the Joint Committee values for equal spans, and, for the case of drop panels particularly, the disagreement is only one per cent or so; in the case of slabs without drop panels it is somewhat greater. Since no data existed for determining whether the critical section for unequal spans is the same distance out from the column, it was decided to try, temporarily, at least, using the same critical section to determine the maximum negative moment.

The method of analysis gives maximum negative and positive moments spread over a section the width of the bay, including what we are accustomed to call the column-section and the mid-section. The next problem was to determine the distribution between columns and mid sections, assuming constant moments in each portion. Mr. Bertin attacked this also, finally arriving at the conclusion that a general solution was too complex to be used for any regulations, so the committee suggests that we use the same distribution between column and mid sections that we use for panels of equal span. This is the same recommendation used by the 1932 German specifications and by the 1937 Pacific Coast Building Officials Conference.

In Sections 1001 and 1002 (a) we have the general assumptions to be made in such an elastic frame analysis. However, a great many of our designs are for equal spans. The 1936 regulations, or their predecessors, covered this case satisfactorily. It was felt that it was not fair to ask the designer to go through the elastic frame analysis each time, when the detailed results were available. Therefore, it was worth while incorporating this special case in these regulations; so, beginning with Section 1002 (b) and including 1003, 1004, and 1005, we have more detailed regulations for the panels of approximately equal spans and approximately square. In general these sections parallel the present 1936 regulations.

Since now the size of the columns is taken into consideration, there are four cases in Section 1002 (b) depending on column size and the ratio of live load to dead load. There are given tables so that we may pick directly from the proper place in each table the positive or negative moment to use in interior and exterior panels. Exterior panels are divided into two cases, (a) where the edge is slightly restrained, (b) where there is a deeper wall beam which gives much greater restraint at the exterior support of the slab. Tables 1 and 2 cover two-way steel with drop panels or without drop panels. In Table 3 minimum lengths of steel for two-way and for four-way steel are given. It was the intention of the committee to include also two tables for moment coefficients for four-way steel, similar to Tables

1 and 2. Somehow in the assembly of this matter, the four-way moment Tables were not printed and I understand that Committee 501 would like to be authorized to include the two omitted tables. The moment coefficients in the tables were obtained by the method I have outlined and compared with the Joint Committee values, and these moments agree reasonably well with the Joint Committee.

This brief statement covers the method of attack made by the flat slab sub-committee on the new and, for us, rather radical development of general regulations for the design of flat slabs, whether they be of equal span or unequal span. We have no doubt that these regulations are capable of improvement as time goes on, but our object is to outline a general procedure. In time a simple method of design will evolve giving safe results with opportunity left to the designer to adapt a general method to a specific problem. We naturally had to make assumptions and study practical methods of design. We tried to keep in agreement with past practice and yet have a general method that would cover unequal spans.

Professor Richart—Is there any discussion of the provision as described to you? This I think is the time to have this discussion. It is probably the most radical change from the 1936 Tentative Code. We certainly should have some comment on this.

Mr. Nichols—It would be possible to discuss a good deal of the detail of what is proposed, but I'd rather not attempt that at this stage. So far as the provision that the first speaker has described as being embodied in the first section, 1001 and 1002 (a), the provision for design by means of the principle of continuity, is concerned, I have no criticism; I think it is an excellent presentation of a positively valuable means for the design of flat slabs, something new. We have not been accustomed to regard flat slabs as continuous frames, and I think we should not confine ourselves too far to that sort of concept until we have had some experience with it. I will explain presently what I mean by "too far." We have here presented a means by which designers may design many flat slabs as a part of a continuous frame. That means giving the designer, for the first time, a method of dealing with flat slabs that are irregular, flat slabs where the panels are too far out of square to fall within the defined limitations, the category of flat slabs, where the adjoining spans are too far from equal. I say here is a method presented for the first time for dealing with such slabs. I will not say that designers have not dealt with such slabs in the past; they have, and with some satisfaction. I mean to say slabs have not, in general, failed; they have, perforce, dealt with such slabs by means of such methods as they could devise. Now I think it is a

very great accomplishment to have this method of dealing with flat slabs, particularly irregular slabs, available to the profession, and a great debt of gratitude is due to Mr. Bertin and his associates who have done an enoromus amount of work in the preparation of this method. The other method of dealing with flat slabs, in which we retain the concept of a floor supported on columns, not a continuous frame, but still a floor, a slab supported on columns; we have had that method of design of flat slabs since 1924, at any rate, and so far as I know it has been a satisfactory method of dealing with what might be called regular flat slabs, not too far out of square and not too far from equal spans. Now while we are gaining experience with the new method, it seems to me that we should not depart too far from the old method. That old method should still be available to designers to use for flat slabs to which they are applicable, that is, to regular flat slabs. To what extent individual designers will stretch the old method to cover irregular slabs is their business, it is not for us to decide. It seems to me that the method presented in this proposed revision, in what comes after Section 1002 (a) departs too far from the old method, and I very much hope that the committee will see fit to return more nearly to the method that is presented in the 1936 Tentative Code. There are some parts of the 1936 Tentative Code that can well be brought up to date, made more useful to the profession without getting very far away from the method used, and I very much hope that we will return more nearly to that before the Code is laid before the Institute for adoption as a standard.

Professor Peabody—Mr. Nichols said that the method was absolutely new. That probably is true for the American Concrete Institute, but it has been specified in building regulations for five years, and I think the Pacific Coast regulations have evolved similar analyses. Flat slabs therefore have been designed under these regulations for five or six years. The assumptions of the German Code are much more brief but more general than the Code we specify here.

Mr. Wheeler—I wish to second what Mr. Nichols has said about this Chapter 10. I do not wish to present a discussion in detail; I should like to call attention to this fact; if you take Table 2, which is given on page 255, for a condition where your ratio of stiffness of column to slab is ten, and design a building accordingly for twenty-foot bays, you will find that you have to increase the reinforcing steel over what we have been using about 19 per cent. You can, however, under this chapter, reduce the thickness of the slab as you proportion it for the actual stresses you find from your analysis. I think that is a step in the right direction, for our slab thicknesses have been too great,

thicker than required, but I do not see that there is any justification, from long experience and numerous tests, for adopting now a regulation which will greatly increase the amount of reinforcing steel.

Professor Peabody—In other words, Mr. Wheeler, you consider that the critical section for negative moment is farther out from the column?

Mr. Wheeler—Well, proceeding on Mr. Bertin's theory, yes, that would be the answer, I think.

Professor Peabody—Do you consider the present 1936 regulations satisfactory? Are they about right?

Mr. Wheeler—They have given satisfactory results, and I believe that the records of tests which are available on flat slabs show that they are certainly conservative.

Dr. Newmark—Some 25 years ago Mr. Nichols pointed out that we have very definite static limitations on the magnitude of the least possible average moment in a flat slab. I believe that that is still the best guide we have to the design of flat slabs. The method now proposed gives a statically possible system of moment also; it must in accordance with the principles of statics and the moments on the various sections add up to a proper amount. However there is no way of determining the distribution of that moment, even though we use the principle of continuity in determining the total moment. We have to do that by guess or by judgment or by experience—call it whatever you will. I should like to raise the question whether it is worth while to proceed with a method of analysis that is tedious and complicated, when, after we finish with that analysis, we still have to guess as to the moment that we design for.

Professor Richart—Are there any further comments? I have the feeling that, had this section been before you a longer time, we would have had a great deal more comment, judging by my own reaction. I am sure I would be more likely to say something about this Code if I had studied it longer.

Mr. DiStasio—I agree with Mr. Nichols; I believe that the 1936 regulations should be maintained in the Code while we try this new method. However, I am not entirely in sympathy with the extent to which this new method goes; in other words, I want to go still further. In Table 2 there is a limitation; the span varies by ten per cent; that means that if you have a slab of 20 feet, the span would be 18 feet or 22 feet. I don't know, with 2 feet variation in the span, that it is worth while to go into this statical analysis. I believe there could be much more variation in span ratios, adjacent span ratios, and if we are going to adopt this method, let us make it practical and let us

adopt the general tables for which those two tables are only the particular portion. I believe that we ought to retain the 1936 regulations and I also believe that this method should have a presentation much more complete, so that we really will have a practical tool. I think a variation of two feet does not mean much; I would not hesitate to design on the old basis on account of a variation of ten per cent or even fifteen per cent, and use my own judgment on the varying moment, so I really believe that this table does not go far enough in view of the radical departure.

Professor Richart—I am not quite clear as to the intent of Mr. DiStasio's criticism. As I understand it, Tables 1 and 2 are intended to cover the usual case, the simple case which was covered in the 1936 Code, and therefore it is restricted to a variation in adjacent span of not more than 10 per cent either way. I understand that the general method which is proposed would cover any kind of variation in span, and should there be a wide variation in span, we would have to use the general method and not the tables. It is almost impossible to make a table which covers a wide range; either you have to have an enormous table or the average values do not mean very much.

Mr. DiStasio—We have a table which, in my opinion, is a lot more practical than this one.

Professor Peabody—I am waiting for Mr. Nichols to rise and say that it is for us to allow the designers to make their own tables. Is that right, Mr. Nichols?

Mr. Nichols—It is very close. I think the provision of this chapter which makes possible the use of the principle of continuity prescribed no particular method of analysis. You may use, if it is available, the method of analysis devised by Mr. Bertin and his associates. You may use what is known perhaps as the Hardy Cross method of distribution of moment. There are possibly a dozen other methods of approach to this problem of the principle of continuity. I am used to calling it the method of elastic analysis; it means the same thing. I am in hopes that Mr. Bertin's method will be made available to the profession through publication in the Institute JOURNAL, but I am very glad that this chapter does not prescribe any particular method. It seems to me we should not unduly restrict the category of flat slabs which we might describe as the regular flat slabs. That category to which the old methods are still to be applicable, and it seems to me that this chapter as proposed now, does unduly restrict that category. I think the field to which the old method may still be applied should cover substantially the field to which they have been applied satisfactorily during the last fifteen years.

*R. A. Caughey**—I notice that on page 254 of the January JOURNAL the moments are given in terms of wl^2. The term w is not defined. What is the relation between w and W? The moment M_o is expressed in terms of Wl; in the table it is also indicated in terms of wl^2. This is not consistent with the current use of these terms in connection with flat slabs, for which W is the total load and w the load per square foot.

Professor Peabody—That is a matter for the Editorial Committee to iron out. I suspect, although I am not sure, that the moment coefficients are written in terms of unit width, since they apply to the column strip and middle strip. In that case I think the notation is correct, but it needs to be defined.

(There being no further discussion Chapter 11—Reinforced Concrete Columns and Walls, was next considered, presented by Professor Richart, with Mr. Stephens taking the chair and no questions or discussion developing, Professor Richart took the chair and Mr. Stephens asked Mr. Bradbury to present Chapter 12—Footings.)

Mr. Stephens—Mr. Bradbury, will you present Chapter 12?

R. D. Bradbury†—Chapter 12 is presented in the proposed revision of the code in its entirety because Chapter 12 has had a complete overhauling. This has been necessary because the requirements for footings have had practically no revision since publication of the old 1928 code. The revisions now proposed were not undertaken with any intention of doing anything radical or making any major changes that would vitally affect current practice in the design of footings, because nothing has come to the attention of the committee that seemed to demand radical changes. However, in reviewing Chapter 12, we found there were a number of inconsistencies and also certain provisions that were subject to different interpretations. For example present Chapter 12 tells us what to do in the case of a square footing and also a rectangular footing in which the length is not more than one and one-half times the width, but it does not tell us anything about what to do in rectangular footings if the length exceeds one and one-half times the width. Also a certain procedure is set up for the computation of bending moments on certain prescribed sections but there is no mention as to how to distribute the reinforcing steel on those sections. Because of such inconsistencies and lack of clearness, the sub-committee on footings undertook to revise this chapter more with the idea of making it readily workable and understandable than with an idea of doing anything radical. I shall touch

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only a few of the high spots in the changes that are proposed. Requirements for determining the unit pressure on the base of footings have been broadened in scope. Instead of always taking the unit pressure on the bottom as the load divided by the area, it is proposed to make provision also for variable pressure in case the footing is eccentrically loaded or the column transmits a moment to the footing. Another feature that is different from present requirements is in regard to computation of the external shear and bending moment on certain prescribed sections. Previously, the external forces with respect to such sections have been considered as acting on a trapezoidal area obtained by drawing lines from the corners of the column to the corners of the footing. In the case of shear computations it is proposed to change this procedure by prescribing that the trapezoidal area be established by drawing from the corners of the columns lines making an angle of 45° with the principal axes of the footing. In the case of a square footing with a square column the present procedure and the proposed procedure are, of course, identical, but in rectangular footings the proposed revision throws a greater load on the resisting sections tributary to the longer projections of the footing. In regard to bending moment computations, it is proposed that the bending moment on any prescribed section be taken as the full statical moment on that section instead of using the moment of the forces acting merely on a partial or trapezoidal area as is now permitted. In other words, it is proposed that the bending moment on any section in a footing be computed from all the forces on one side of the section and with the section extending completely across the footing. Obviously, this procedure will give slightly greater bending moments than we have been in the habit of using. There can be no argument as to the correctness of doing this because, regardless of the shape of the footing or the position of the column, the total bending movement on a given section is equal to the moment of all forces acting on one side of that section. While such a computation gives the numerical value of the total bending moment, it does not, however, tell us anything as to the distribution of that moment across the section when we have moments acting in two directions at right angles to each other. However, this is not of great importance in our case because it affects only the distribution of the tensional steel and, as a matter of fact, we have been in the habit in the past of assuming an arbitrary distribution of the steel and in addition have been inconsistent in using only a part of the external forces in computing the moment. This question of using the full statical bending moment in footings developed considerable discussion in the com-

mittee. The discussion, however, did not hinge upon the correctness of using the full statical moment but upon the question of what effect it would have upon the practical design of footings and how it would affect current practice. It so happens that the bending moment computation in footings is important only in regard to the reinforcing steel. The reason for this is that, with the limit which has been placed on the allowable unit shear when used as a measure of diagonal tension by permitting a maximum unit shear of 75 pounds per square inch regardless of the strength of concrete used, the depth of the footing in practically every case will be determined by shear and not by bending. Adopting the statical moment method of computing bending moments will, of course, result in slightly larger moments than we have been using and, if full tensional resistance is provided in the steel, we will require more steel in our footings than we have been in the habit of using in the past. That is not a desirable result because there appears to be no practical reason why steel in footings should be increased. In regard to this phase of footing design I have personally interviewed a number of practicing engineers and none of them feel that more steel should be used in isolated footings. As a matter of fact, I find many who feel that too much steel is being used now. It therefore seemed to the committee that we have no practical justification for deliberately increasing the amount of steel in footings and the committee, in order to harmonize the new provision with established practice, proposes that the moment of resistance provided by the steel be taken as 15 per cent less than the computed statical moment. This is an arbitrary reduction but it was adopted after the committee made a great number of computations for varying conditions and the suggested reduction was arrived at as being that which would satisfactorily harmonize final results in line with current practice. While this 15 per cent reduction is purely arbitrary, still we feel that it is extremely conservative, especially when it is realized that, in the flat slab section of the code, an arbitrary reduction of about 25 per cent is permitted. Hence, the footing chapter is not nearly as radical in this regard as are other parts of the code. For the rest of Chapter 12 there is no need of going into detail in regard to all of the proposed revisions. I have touched only upon the more important high spots. As Chairman of the Sub-Committee on Footings I feel somewhat as Prof. Richart does in regard to columns. There are a few minor points in the proposed revision of Chapter 12 that we would like to change further for the sake of clarification and in some cases for conformity with other requirements that are prescribed. These, however, are not vital but, if included,

will assist greatly in dressing up the chapter and making it more usable and more consistent.

Mr. Nichols—In Sec. 1204 (d) there is a provision for design of tensile reinforcement in one-way reinforced concrete footings which provides that tensile steel shall be designed for the full moment of resistance. Now one-way footings are footings under walls, possibly under masonry walls or concrete walls. In the next paragraph there is provision for the design of tensile steel in two-way footings, and in two-way footings this fifteen per cent reduction is allowed. I would like to ask why the reduction is allowed in the case of two-way footings and not in the case of one-way footings?

Mr. Bradbury—That resulted from general discussion in our committee. There was a difference of opinion whether or not to continue to use the segmental method, or the full section method. The full section method, finally prevailed, but there was a feeling that one-way footings which, as you say, are usually wall footings, should be designed in the same manner that has always been the practice. In other words, it was consistent with our policy to try to harmonize the requirements and get results comparable to those now being obtained.

Mr. Nichols—As I understand it, it is just arbitrary?

Mr. Bradbury—Purely arbitrary. The amount arrived at was, as I say, the result of trying to harmonize results obtained by the old method and the new.

Mr. Nichols—I have been unable to see any reason why the reduction should be made in one case and not the other.

Mr. Bradbury—I agree with you absolutely, Mr. Nichols. In years gone by I have heard many times about this reduction factor. In my opinion the reduction, if substantiated at all, is substantiated on the basis of test. Even up to the ultimate load, the steel is never brought completely up to its theoretical; there is a large residual tension in the concrete or something; we base it on that as a port of refuge. Now the reduction, as I say, was made arbitrarily in order to prevent using excessive amounts of steel as compared with what is now currently used.

Mr. Nichols—May I suggest that there are possibly three explanations for reductions. Perhaps the upward pressure of the soil on the footings is not fully vertical; perhaps it heads inward toward the center of the column or toward the center of the wall. Now the second is that perhaps it is not uniform under the footing; possibly the pressure may be greater at the edge than under the center, but if it were believed

that it would be greater under the center or that the direction of the pressure would be toward the center of the wall, there were two reasons possibly for making this reduction, two reasons for justifying the equally empirical method of the trapezoidical area. And the third reason for making this reduction is that tension exists in concrete, in slabs and rectangular beams and footings which give assistance to the tensile steel. I do not know of any other possible explanation. I will not say that I do not believe any other explanation is possible, because I am not omniscient, but if there is no other explanation, it is perfectly clear that this reduction ought to be made in the case of one-way footings.

Professor Richart—Gentlemen, I dislike very much to curb discussion at this time. On the other hand, we want to get some action on the proposed revision of the Code and I would like to ask now that we withhold further discussion, that you submit it as written discussion later and that we do proceed with the final action on this revision. I am sorry that we are so crowded for time.

Mr. Stephens—I have now presented all the chapters. We have not voted on several of the chapters. We have voted affirmatively on a few. What is your pleasure?

Nr. Nichols—In view of the fact that time is lacking for consideration of all the changes that are proposed and some of them are quite radical, I would like to move that notwithstanding the action taken yesterday on a few chapters, the entire proposed Code be referred back to the committee 501. (The motion was seconded.)

Professor Richart—You have heard the motion, gentlemen. I think it has been apparent throughout the discussion that there were many points which need clearing up. This committee has worked hard on this particular assignment since 1936. I don't know how many meetings it has held in that time. The members are scattered pretty well over the country and it has been difficult to get full meetings. The committee has done excellent work under difficulties. I think in referring the matter back to the committee for further polishing up and improvement, it is no reflection on the action of the committee. You have the motion before you. Is there any discussion? The motion was unanimously adopted.)

Professor Richart—The motion is carried. The matter is referred back to the committee for further attention and it is hoped, of course, that it will be presented at the next annual Convention and that in the meantime it will be printed well in advance of the Convention and that there will be articles published illustrating some of the new

developments which are contained in the revision and that it will come to the next Convention much better prepared for adoption as a standard.

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A Questionnaire on Concrete Vibration*

By Arthur Ruettgers†
CHAIRMAN, A. C. I. COMMITTEE 609, VIBRATION OF CONCRETE

SYNOPSIS

Chairman Ruettgers, in laying a foundation for the further work of his committee in the study of and recommendations for the use of vibration as a means of compacting concrete, devised a questionnaire of many items. To the popular "true or false" reply he provided for adding another element to the quiz in an opportunity for registering the opinion that a question needed further investigation. The tabulated results are published as of likely general interest in showing the extent of agreement and of disagreement on important phases of the subject.

As an initial step in planning the current activities of Committee 609, it was thought that there should be secured from the committee members an expression of their views with the object of developing a nucleus of generally accepted findings or truths around which to build. Such a canvass, it was considered, would also focus attention on controversial subjects, and on questions which needed further investigation. A questionnaire was therefore distributed to the committee membership, containing statements to which the members were invited to agree, disagree, or signify that in their opinion the subject required further study.

The interest manifested in this questionnaire and the nature of the comments submitted by some members suggested that it would be desirable to present a digest of the committee's reaction for the general information of the Institute. A second purpose in publishing the digest is to invite discussion that may be helpful to the committee as well as to JOURNAL readers generally.

*Received by the Institute, Nov. 3, 1939.
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The various statements of the questionnaire, together with a tabulation of the committee's vote thereon, are presented. It may be observed that the total vote on each statement is not always the same. The explanation is that in some instances the voting was incomplete, while in other cases certain members indicated, in addition to expressing agreement or disagreement with a statement, that there was need for further investigation of the question involved.

QUESTIONNAIRE ON CONCRETE VIBRATION

*In the first column of figures at right is given the number of votes of committee members agreeing to each proposition; in the second column, those disagreeing, while the third column records the opinions in favor of further investigation.	Vote of Committee Members*		
	Yes	No	?
1. Concrete of drier consistency may be satisfactorily compacted by vibration than by ordinary tamping methods	17	0	0
2. Concrete slumps suitable for use under ordinary tamping methods of compaction may be reduced approximately 50 per cent by the effective use of vibration.	14	2	2
3. Axiomatically, the use of vibration permits the use of lower water-cement ratio for concrete of the same mix proportions, or of a lower cement content for concrete of the same water-cement ratio.	17	0	1
4. The use of vibration generally permits the use of a higher proportion of coarse aggregate.	16	0	4
5. For concrete of plastic consistency and fixed water-cement ratio, vibration as normally and intelligently practiced:			
a. Does not materially affect strength.	11	2	4
b. Does not materially affect shrinkage.	9	1	7
c. Does not materially affect elastic and plastic properties.	7	3	8
d. Increases unit weight by decreasing the air voids.	14	3	1
e. Increases watertightness.	15	1	3
f. Reduces honeycomb.	16	1	0
g. Increases pitting on formed surfaces.	11	3	6
h. Improves bond at construction joints.	15	0	3
i. Improves bond with steel and embedded parts.	14	0	3
j. Reduces the labor requirement and generally the total cost of compaction.	10	2	4
k. Requires the use of tighter and more rigid forms.	17	0	1
6. For concrete of given mix proportions, any reduction in water-cement ratio through the use of vibration will serve generally to extend and accentuate the improvements in physical properties detailed under item 5.	13	0	3
7. Overvibration of plastic concrete is conducive to internal segregation or disarrangement of the constituent materials, with resultant reduction in uniformity of the concrete.	16	0	1
8. There is no particular advantage to be gained by vibrating sloppy concrete; on the contrary, such vibration is likely to cause objectionable bleeding and other evils associated with overvibration of plastic concrete.	16	0	0

	Yes	No	?
9. Vibration should not be relied upon to correct segregation, and particularly lateral segregation of mortar from the coarse aggregate.	17	0	0
10. Extensive lateral movement of concrete by means of vibration is conducive to objectionable segregation and nonuniformity.	15	0	2
11. The strength and bond of concrete are not adversely affected by revibration applied within a reasonable time after placement of the concrete.	11	0	6
12. Pitting on formed concrete surfaces is wholly attributable to entrained air.	8	7	5
13. Present means for judging the sufficiency of field vibration by surface appearance are inadequate, especially when form or surface vibrators are used, or when there is lack of assurance, in using internal vibrators, that each successive increment of concrete deposited has been penetrated to its full depth.	13	1	6
14. There is need for some simple device to measure the effectiveness of vibrators under field operating conditions.	14	2	7
15. Internal vibration is generally superior to external vibration.	17	0	0
16. Horizontal vibration (referring to the plane of the motion causing the vibration) is generally more effective than vertical vibration.	8	0	7
17. The transmittal of vibration to concrete diminishes with the distance from the vibrating element, and as there is a limit to the range of effectiveness, some overlapping in coverage areas is necessary to insure adequate compaction of the entire mass.	17	0	4
18. There is need for more basic information for evaluating the effects of frequency, amplitude, acceleration, and period of vibration.	17	0	4
19. Vibration acceleration is directly proportional to the amplitude and to the square of the frequency.	7	2	7
20. Within limits of frequency and amplitude, the effectiveness of vibration is probably more intimately related to acceleration than to either frequency or amplitude.	3	2	9
21. For a given concrete and set of conditions there is an optimum acceleration or combination of frequency and amplitude for most effective compaction.	7	0	9
22. Each concrete and set of conditions probably has its own optimum acceleration or combination of frequency and amplitude.	6	2	9
23. For concretes in general there is a minimum frequency below which vibration is relatively ineffective.	17	0	0
24. For concretes in general there is a maximum frequency above which the vibrator upkeep costs are excessive.	14	1	3
25. For a given concrete and set of conditions, there is probably a maximum frequency and a maximum amplitude above which vibration becomes relatively ineffective or actually harmful.	5	1	9
26. Vibration is produced in most vibrators by rotary motion of an eccentric weight, and high frequency vibration is more easily achieved by rotary motion than by reciprocating motion.	13	0	3
27. Internal vibrators in which the motor and case are directly attached to a vibrating element of considerable weight are best suited for vibration of mass concrete.	10	2	3

	Yes	No	?
28. Internal vibrators with flexible operating handles are best suited for vibrating concrete within enclosed forms and around reinforcement.	13	3	0
29. There is pressing need for adequate specifications and supplemental information for the guidance of those concerned with the purchase and use of vibrating equipment.	16	0	2
30. Before such adequate specifications and supplemental information can be intelligently prepared, it is necessary or desirable that more data of a fundamental character be obtained from laboratory tests under controlled conditions.	16	0	1

In connection with statement No. 5, it should be noted that the wording of the introductory clause admits of two conditions or cases—one in which identical mixtures are used for the two methods of compaction (vibration and hand), and the other in which the mixture for vibration compaction is drier and consequently leaner (to maintain the water-cement ratio) than the mixture for hand compaction. Unfortunately the substatements (a) to (k) were framed without keeping both conditions in mind and the responses of the committee members were influenced by their interpretations of the questions. This is particularly true of substatement (b), to which it is felt a substantial majority would have subscribed had it been made to apply to identical concrete mixes. A number of the members stated, quite properly, that reduced shrinkage may be expected when mixes for vibration compaction are made drier and leaner, the extent of the reduction in shrinkage being governed by the reduction in cement paste content.

Members of Committee 609 were also requested to give their opinions on several questions which, by their nature, required more extended answers than the statements in the questionnaire. A few of these are quoted and a digest of the replies is given for the benefit of the reader.

(1) Assuming the availability of electric current and compressed air, what preferences do you have between air, electric, and gas power vibrators for various climatic conditions and classes of work, and what are the reasons for your preferences?

Seven members favored electric power-driven vibrators, three gave air power as their first choice, and one stated that there was little choice between electric and air-driven equipment. The opinion that gasoline-driven vibrators were the least desirable was quite unanimous. One member, who preferred air power equipment under ordinary conditions, considered the electric drive more desirable during cold weather. The disadvantages of air equipment due to freezing of

condensed moisture was cited by several, but two members defended this type of vibrator vigorously, stating that freezing may be overcome if antifreeze agent is trickled into the air line. Air-driven vibrators were considered by their proponents to be more effective than the electric in large sizes, to be generally lighter, and to require less maintenance.

(2) What forms of investigational work do you think should be undertaken (in addition to those indicated by your responses to the list of statements) to assist in making the committee's former recommendations on the use of vibration more accurate, more explicit, and more complete?

The answers to this question generally indicated the desire for a method of determining the efficiency of vibrators, the factors which influence efficiency, and the determination of a proper measure of vibration. The causes of and means for avoiding pitting and sand-streaking with vibration, and the danger and hazards of overvibrating, were also urged as subjects for study.

(3) Should the committee include in its work the development of specifications for use in the purchase of concrete vibrators? If not, how far should the committee proceed in providing information to guide the prospective purchaser in the procurement of equipment best suited to his requirements?

The general reaction to the first part of this question was in the affirmative. However, the majority of the members also expressed the opinion that specifications should not be too specific or detailed but should be made to cover the essentials for getting the proper type of vibrator for each class of work. A guide to purchasers was also suggested which would include a discussion of the merits and disadvantages of the various types of vibrators and their most appropriate uses. Some members were in favor of performance tests only and stressed the desirability of some method for measuring the effectiveness of vibrators.

The current work of the committee is implied in the answers to the last two questions and in the number of requests for further investigation applying to the statements in the questionnaire. Subcommittees have been formed, assignments of work have been made and accepted, and much of the work is in progress. It is expected that within a year the committee will have positive information to enable it to submit a revised report* embodying specifications and recommended practice for the purchase and use of concrete vibrators.

*For previous report of Committee 609 giving "Recommendations for Placing Concrete by Vibration," see March-April 1936 issue of the JOURNAL (*Proceedings*, Vol. 32, p. 445). (This report available also in separate pamphlet.)

Discussion, to close in June, 1940 JOURNAL, should reach A. C. I. Secretary in triplicate by Apr. 1, 1940.

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Concrete Aggregate Development on the Claytor Hydro Project*

By G. W. Hutchinson†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

This paper describes in summary the development of both fine and coarse aggregate for concrete on the Claytor Hydro Project, near Radford, Va. It covers studies of particle shape and gradation which resulted in the development of a stone sand of high quality and enhancement in the value of the coarse aggregate as compared with the ordinary commercial product usually secured from primary crusher operation. Data are presented to support the departure from somewhat orthodox procedure in the use of relatively large amounts of the dolomitic aggregate passing a No. 100 sieve, in concrete of relatively low cement content in the mass, or unexposed work. The saving of several hundreds of thousands of dollars on concrete construction through these developments was possible.

INTRODUCTION

The Claytor Hydro Project is on the New River in Pulaski County, Virginia. It consists of a gravity type of dam and a power house, both of which involved the placing of about 250,000 cubic yards of concrete. Approximately 75 percent of the concrete was unexposed in the dam and was that of the lower cement content referred to in this paper as mass concrete. The remainder, all exposed, both plain and reinforced, has the higher cement content. This included a face mix about three feet thick encasing the mass concrete. The dam is approximately 1150 feet long, having a maximum width at the base of 108 feet and a maximum height from rock to the top of the spillway gates of 123 feet. The power station includes four generating

*Received by the Institute Sept. 13, 1939
†Concrete Technician Claytor Hydro Project

units, each with a rated capacity of 26,000 horsepower. The reservoir is 22 miles long, covers 7 square miles and has a shore line of 100 miles. Concrete construction began September 1, 1937 and was completed by June 1, 1939.

It is the purpose of this paper to describe the more important developments which affected the original design of concrete and dictated subsequent changes during construction. The development of a high quality crushed stone sand; reshaping certain sizes of coarse aggregate; and the somewhat judicious use of minus 100 dolomitic aggregate, as judged by orthodox tendencies, resulted in saving several hundreds of thousands of dollars and contributed to improvement in the quality of concrete as construction progressed.

PRELIMINARY TESTS OF AGGREGATE

Research of preliminary nature was carried on several years before construction started. An existing quarry at the dam site had been purchased to provide coarse aggregate for the concrete and limited tests were made later to determine the suitability of the same material as fine aggregate. Failure to develop such fine aggregate would have made it necessary to transport natural sand for more than 200 miles.

Preliminary tests of mortars were made to compare the material of sand size then produced by the quarry with the nearest available source of acceptable natural sand. This consisted of primary crusher particles plus a small amount of oversize material recirculated through a ring roll crusher of low capacity. The results indicated a slight superiority to the natural sand by the usual tensile, compressive, and freezing and thawing tests. No studies of workability were made. The data justified further study in the development of stone sand as a matter of economy and quality.

Just prior to the beginning of construction, research was undertaken in connection with the design of concrete with the idea that the quarry would furnish all fine and coarse aggregate.

Studies were then made to determine the effect of particle shape on workability of mortars. Through lack of other means, the flow table was necessarily adopted as a measure. To provide sufficient fine aggregate supply, it became necessary to purchase additional crushing equipment. Stone of various sizes was furnished to manufacturers of different types of crushing equipment for reduction to sand size as the basis of a study of particle shape. This in turn was to provide a basis upon which to purchase the additional equipment.

Although the desirability of the so-called cubical as compared with the flat particle had been realized for some time in connection

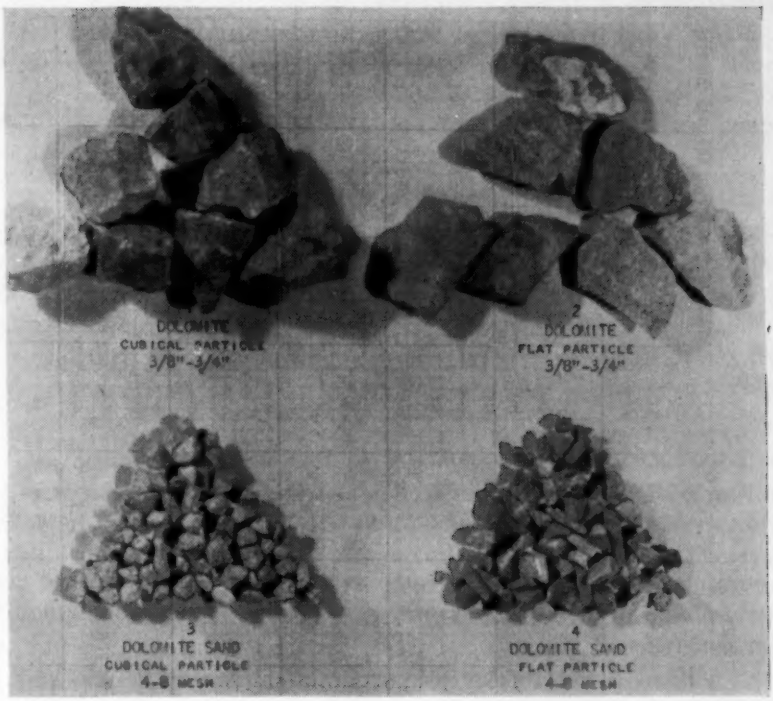


FIG. 1—COMPARATIVE PARTICLE SHAPE FROM TWO TYPES OF CRUSHER EQUIPMENT

with the effect on workability, definite data were limited. Except for Norris Dam, then just completed, there were no examples of quantity production of modern mass concrete using crushed stone as both fine and coarse aggregate and placed by internal vibration.

Considering the material secured with this rock from the primary crusher as the flat particle, the product of the ring roll crusher gave the nearest approach to the other extreme, or cubical material, in its effect on the placeability of concrete. This type of crusher was subsequently adopted, being supplemented by a rod mill for the reduction of the smaller sizes. An idea of the two extremes may be obtained from Fig. 1.

Failure to secure sufficiently prompt delivery of the crushing equipment prevented full operation of the sand plant until after construction had started. This necessitated the initial use of a large portion of the product of the primary crusher—a material of poor shape. This was gradually replaced by the improved product

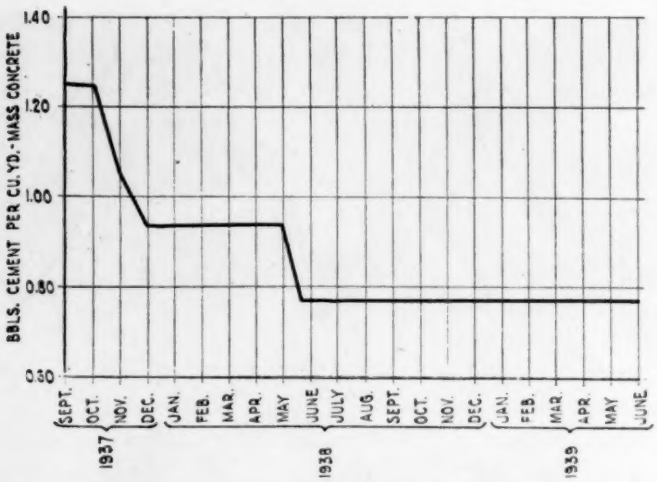


FIG. 2—DECREASE OF CEMENT IN THE MIX WITH DEVELOPMENT OF THE AGGREGATE

as equipment for making it become available. The handicap of poor article shape of fine aggregate was overcome about three months after construction started.

Regardless of theoretical proportioning and mix design and in some cases regardless of laboratory indications, the main consideration throughout construction was the quality of concrete after it had been vibrated into the forms. During the early period, it was impossible to satisfy this consideration without the use of relatively high cement contents. Even though these greater cement contents provided relief to a large extent in placing the concrete properly, cracks, especially parallel to the longitudinal axis of the dam, shortly became apparent. This stressed the fact that increased workability through improved particle shape of the aggregate was necessary to allow the use of lower cement contents and with less consequent heat generation and volume change. The average compressive strength of the mass concrete for the entire project was approximately 3500 p.s.i.

Decreases in the cement content of the mass concrete were dependent entirely upon increases in workability derived from improved particle shape and gradation of the aggregate. The latter included adjustments in the amount of minus 100 dolomitic aggregate. A general picture of the progressive decrease can best be visualized by referring to Fig. 2. It indicates graphically the average cement content of the mass concrete which was used to maintain proper workability throughout construction of the project. Hydrau-

lic lime was added in various amounts. The total paste content, cement plus hydraulic lime, on a weight basis, approached a maximum in the early period of construction equal to over 1.50 bbl. of cement per cu. yd., and was decreased gradually to a minimum of 0.77 bbl. of cement with an addition of 10 percent (by weight of the cement content) of hydraulic lime.

The fact that both fine and coarse aggregate were made from the same stone simplified portions of the problem. For convenience, test results include comparison of but two particle shapes. They constitute the extremes secured on this particular project. Other research indicated that workability, water cement ratio, etc., were affected directly in proportion by any particle shape between these two extremes, as well as by any combination of the two extremes themselves.

The ultimate and relatively low cement content of the mass concrete would not be recommended for universal adoption without thorough investigation of the materials at hand and the establishment of the fact that proper placement could be secured. The aggregate consisted of high quality dolomite having a sound uniform structure. The quality is indicated by the following tests:

			Percent
Specific Gravity:	2.80	Silica:	21.52
Absorption: (Ave. ⅜"-3")	0.5%	Iron and Alumina:	2.60
Percent of Wear:	4.44	Calcium Carbonate:	43.10
Cementing Value:	33	Magnesium Carbonate:	32.64
Toughness:	32	Undetermined:	0.14

The quality of fine aggregate made from this stone may be judged by typical routine tests given in Table 1 and Fig. 3.

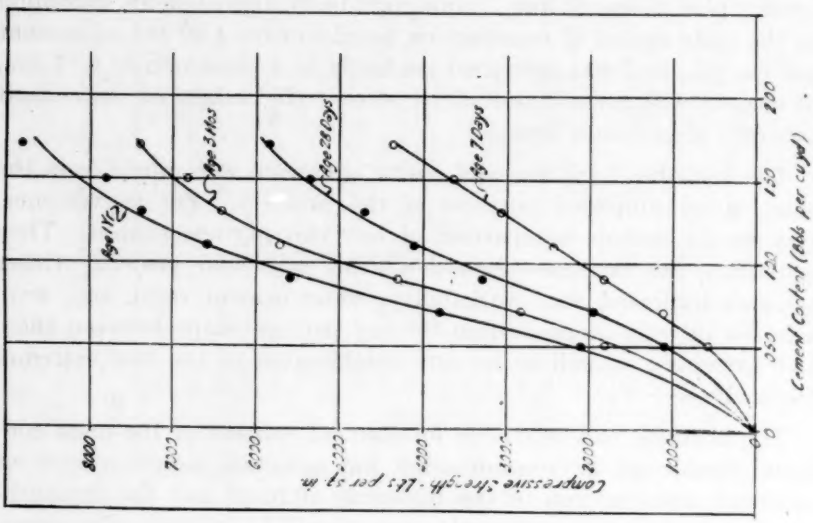
TABLE 1—RELATIVE TENSILE AND COMPRESSIVE (2 x 4 CYLS.) TESTS OF CLAYTOR HYDRO PROJECT RESHAPED DOLOMITIC STONE SAND

Series: F-492
Consistency: Flow 25
1 : 3 Mortar (Wgt.)
Age: 7 Days—HES Cement

RELATIVE STRENGTH

	Stone Sand		Standard Sand	
	Tension	Compression	Tension	Compression
Average of 11 gradations	169	171	100	100
1 gradation giving max. strength	230	202	100	100

Regardless of the quality of aggregate and the degree of workability possible with extremely lean mixtures, the minimum cement content in exposed plain concrete was 1.20 bbl. and the maximum in the reinforced structures was 1.70 bbl. per cu. yd. of concrete. It varied with the quality of concrete required in the reinforced and



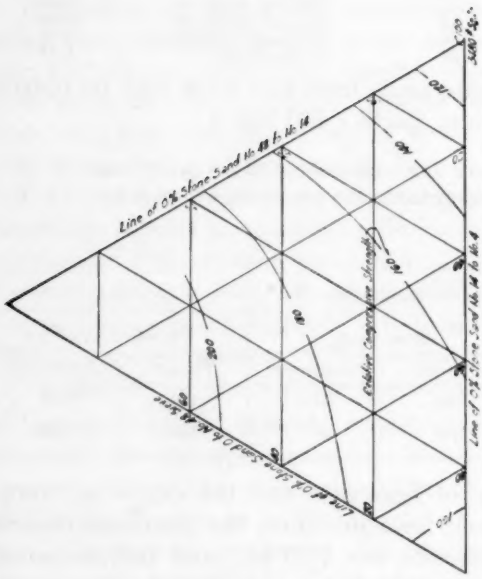

Line of 0% Stone Sand No. 40 to No. 14
Line of 0% Stone Sand No. 14 to No. 4
Line of 0% Stone Sand No. 4 to No. 4 Stone
Relative Compressive Strength

FIG. 3 (ABOVE)—SEE ALSO TABLE 1 AS TO QUALITY OF STONE SAND

FIG. 4 (RIGHT)—BASIC COMPRESSIVE STRENGTH CURVES FROM LABORATORY CONCRETE—EARLY IN THE INVESTIGATION

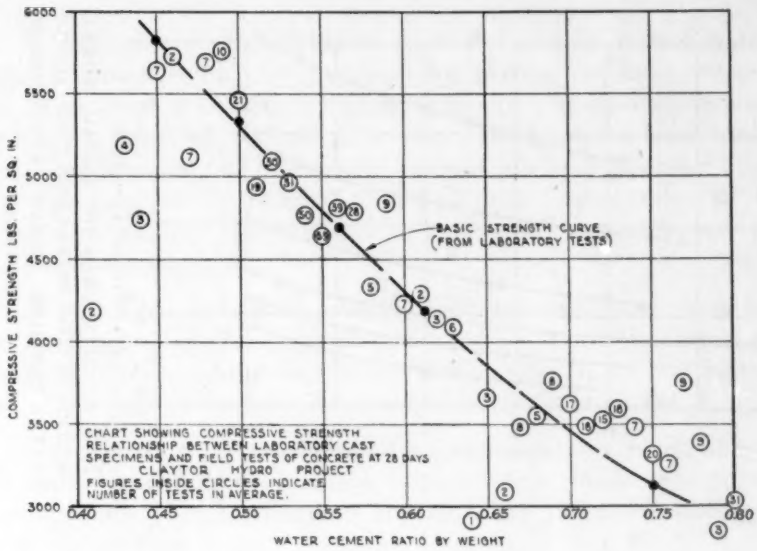


FIG. 5

exposed sections. A maximum water ratio was set for each class of concrete. The various classes of concrete are tabulated below:

Class of Concrete	Ave. Cement Content Bbls. per cu. yd.	Ave. WCR by Weight
AA	1.65	0.44
A	1.50	0.53
B	1.40	0.53
C	1.20	0.56
D	0.77	0.68*

*Based upon total weight of portland cement and hydraulic lime.

Basic compressive strength curves from laboratory concrete were established during the early period of construction. These are shown in Fig. 4. The investigation consisted of 280 6 x 12 in. cylinders using 1½ in. as the maximum size of aggregate and a consistency to allow thorough compaction by high speed vibration. Agreement between the compressive strength of concrete taken from the mixers and the laboratory results, represented by tests at the age of 28 days, is indicated by Fig. 5. Similar agreement was found in the 7-day and in such 1 year tests as have been completed to date.

Both the flow table and slump test were inadequate as a measure of relative placeability of concretes having dry consistencies and low cement contents. This resulted in the development of an appara-

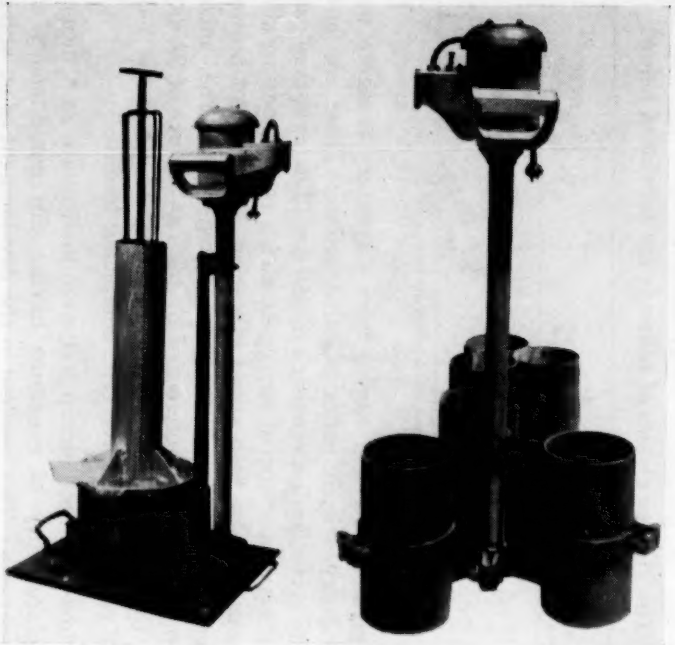


FIG. 6 (LEFT ABOVE)—APPARATUS TO MEASURE PLACEABILITY

FIG. 7 (RIGHT ABOVE)—APPARATUS TO APPLY VIBRATORY ACTION IN MAKING TEST SPECIMENS

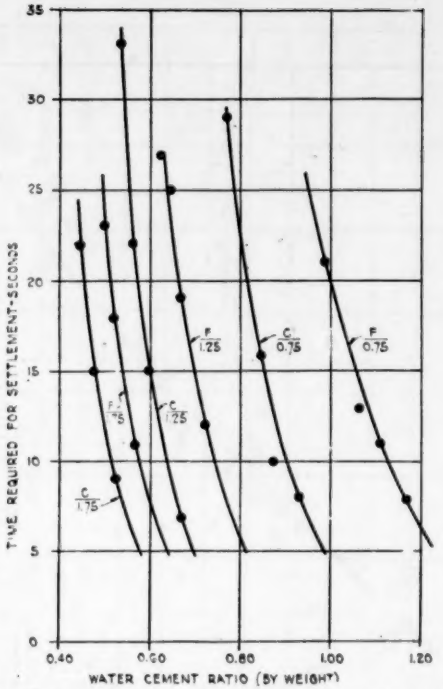


FIG. 8—MIXTURES OF EQUAL WORKABILITY BUT OF DIFFERENT FINE AGGREGATE PARTICLE SHAPE AND CEMENT CONTENT

F = flat; **C** = cubical. Below these symbols in the diagram the figures give cement content in bbl. per cu. yd.

tus to measure the placeability of such mixtures by means of high speed vibration similar to that used for placing the field concrete. The apparatus* is shown in Fig. 6. The time, in seconds, required for a given batch of concrete to be converted from the loose freshly mixed condition to one of ultimate compaction by high speed vibration was used as the measure of placeability. Appearance of the concrete, as regards bleeding, mortar sufficiency, etc., after test was a valuable consideration.

The standard method of rodding both the laboratory and field test specimens was also inadequate for correct representation of the concrete in place. Vibratory action was developed for the fabrication of the test specimens. This apparatus† is shown in Fig. 7.

In summarizing the more important investigations which affected the ultimate design of concrete, separation is made into several classifications. For convenience the data are presented under the following headings:

1. Fine aggregate particle shape and grading tests.
2. Coarse aggregate particle shape.
3. Tests to determine the effect of minus 100 dolomitic aggregate.
4. Effect of cement on placeability.

1. *Fine Aggregate Particle Shape and Grading Tests.* Fig. 8 indicates the basis upon which mixtures of equal workability, but of different fine aggregate particle shape and cement content, were determined. The two fine aggregates are identified by the letter "F" for the flat, and the letter "C" for the reshaped cubical particle. Below these letters is given the cement content in terms of barrels per cubic yard of concrete.

Fig. 9 indicates the compressive strength of concretes made with these consistencies and particle shapes. The fine aggregates were graded from 0 to No. 4 sieve and the coarse aggregate (reshaped dolomite) from $\frac{3}{8}$ in. to $\frac{3}{4}$ in. Fine aggregate constituted 50 percent of the total aggregate content. It was separated into sizes 0-48, 48-14, and 14-4 in each case, and was recombined to give identical gradings. No attempt was made in this investigation to adjust the fine/coarse aggregate ratio so that the optimum, or most efficient combination, as affected by both particle shape and cement content, would be used. Additional tests, not reported in this paper, indicated that such correction, although desirable for refined design, would not affect these particular results appreciably. The data in

*Described in *Bulletin* American Society for Testing Materials, Aug. 1939.
†Described in *Concrete*, Aug. 1939.

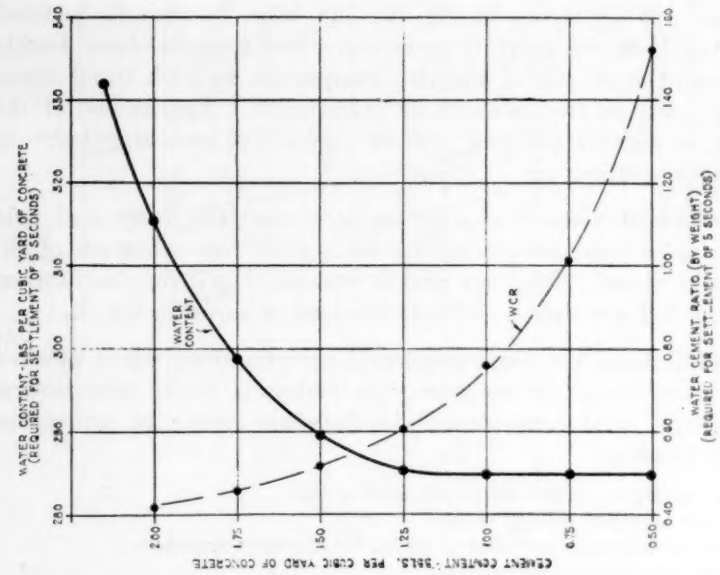

WATER CONTENT-LBS. PER CUBIC YARD OF CONCRETE (REQUIRED FOR SETTLEMENT OF 5 SECONDS)
WATER CONTENT
W/C R
WATER CEMENT RATIO (BY WEIGHT) (REQUIRED FOR SETTLEMENT OF 5 SECONDS)
CEMENT CONTENT-BBLS. PER CUBIC YARD OF CONCRETE

FIG. 10—SEE TEXT, OPPOSITE PAGE

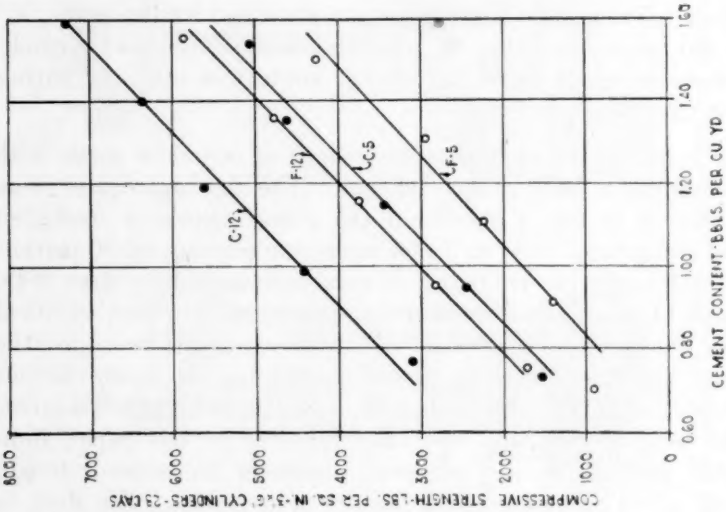


FIG. 9—COMPRESSIVE STRENGTHS OF CONCRETE WITH VARIOUS CONSISTENCIES AND PARTICLE SHAPES. (SEE TEXT PREVIOUS PAGE)

the figures referred to represent one of the reasons for discarding the flat particle of the primary crusher in favor of the reshaped material.

In this particular investigation, a time of settlement of 12 seconds was considered suitable for the consistency of concrete most practical for placement by high speed vibration in ordinary masses and larger reinforced sections. The time of settlement of 5 seconds was suitable for locations where the wetter consistencies were necessary. Mixtures which settled in 5 seconds can best be described in orthodox terms as having a slump of about 3 in. Those represented by the 12 second time of settlement would be on the border line of having no measureable slump.

The amount of water necessary to establish mixtures of equal placeability with given aggregates but with different cement contents is shown in Fig. 10. The total water content for the leaner mixtures is essentially the same, but, as the cement content is increased beyond a given amount, depending upon the gradation and other characteristics of the aggregates and cement, an increase in the total water content is necessary to maintain a given placeability.

A study of particle shape of fine aggregate in connection with freezing and thawing tests offers the interesting data given in Fig. 11. The materials used are identified as follows:

"FD" represents the flat dolomitic fine aggregate from the primary crusher.

"D" represents the reshaped particle from the same stone.

"L" represents a limestone which was present at one end of the quarry adjacent to the dolomite used in construction.

"PR" represents a sample of Potomac River sand used for comparison.

Each curve, with the exception of that for the Potomac River sand, represents the average loss in weight, by freezing and thawing, of eighteen 2 in. mortar cubes—three each of six different gradations of aggregate from 0 to No. 4 sieve. Six 2 in. mortar cubes—three each of two different gradations—are represented by the curve identified as "PR". The consistency (Flow: 80) and proportion of cement (1 : 3 wgt.) were the same for all mixtures. The difference in water contents necessary to produce this consistency, as judged by the flow table, was somewhat less than that required with the same fine aggregate in concrete when measured by the placeability machine. It might follow, therefore, that when more accurate means of determining placeability were used, a greater difference in values would be encountered. There is an appreciable difference in the resistance to freezing and thawing between the same dolomite sand with varia-

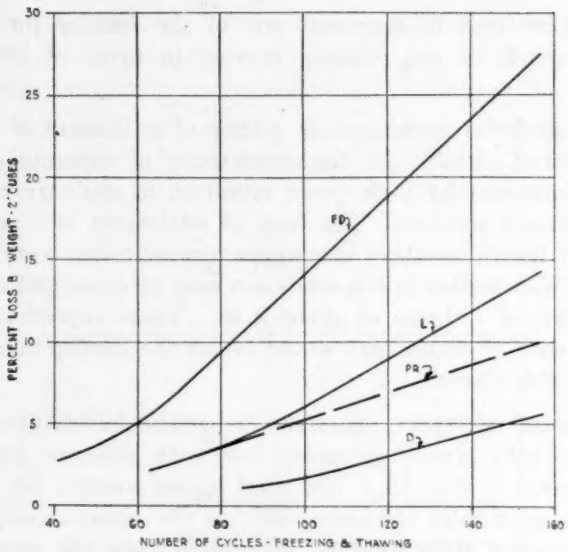


FIG. 11—DATA FROM STUDY OF FINE AGGREGATE WITH FREEZING AND THAWING TESTS (SEE TEXT PREVIOUS PAGE FOR IDENTIFICATION SYMBOLS FD, D, L AND PR)

tion in particle shape. The limestone sand "L", although reshaped by the same process as the dolomitic sand, "D", indicates a higher loss on freezing and thawing. Previous investigation and subsequent experience during the placing of concrete in which both the dolomite and limestone sands were used furnished sufficient evidence for the belief that our limestone was not inferior in quality to our dolomite, but that, with the particular crushing machinery used in manufacture, the limestone did not crush to equivalent efficiency with respect to particle shape. Improved particle shape, which automatically provided for lower water ratio, rather than quality of stone, was judged to be largely responsible for the greater resistance during an equal number of freezing and thawing cycles. For this reason, dolomite was used as much as possible throughout construction of the project. At times, when certain amounts of limestone aggregate would be present, a decrease in placeability in the field concrete was obvious.

Also, a portion of the dolomite, immediately adjacent to the limestone in the quarry, fractured inefficiently and produced essentially the same effect on placeability of the concrete as the limestone.

2. *Coarse Aggregate Particle Shape.* Study of particle shape resulted in two requirements affecting the coarse aggregate. The first

established the minimum size at ⅜ in. In the normal primary crushing operation, quality, as regards particle shape, is somewhat proportional to the size of aggregate, ie., the smaller sizes tend to be flat and the product becomes more cubical with increasing size. By limiting the minimum size of coarse aggregate to ⅜ in., the poorest element, or that graded from No. 4 to ⅜ in., was eliminated and became available for reduction to make part of the fine aggregate. Study of the particles larger than ⅜ in. resulted in recirculation during the crushing operation so that a greater portion, which was of 3 in. maximum size, consisted of reshaped material between ⅜ in. and 1½ in. A supply of aggregate passing a ¾ in. and retained on a ½ in. screen was handpicked to secure the extremes in both the flat and cubically shaped particles. There were approximately 66 particles of the cubical material and 115 particles of the flat material per pound.

Placeability tests were made using the reshaped dolomitic sand as fine aggregate and each type of this handpicked material as coarse aggregate. The difference in water content for these mixtures, with all factors equal except particle shape of the aggregate between ½ and ¾ in., is indicated by Fig. 12.

Many of the data showing the difference between these extremes in coarse aggregate, in terms of water cement ratio when in concrete, are given in Fig. 13. The concrete used in making these tests was afterward cast into 6 x 12 in. cylinders by the vibratory means described for making test specimens. The concrete was compacted as much as possible by vibration, but the relative placeability was obvious by inspection of the specimens after removal of the molds. Compressive tests of these specimens at the age of 3 months offer an interesting indication of the quality of concrete secured where attempts are made to use water contents which are too low. The results show (Fig. 14), the usual contradiction of the water cement ratio theory where the mixtures are too dry to be properly vibrated into place, and a definite tendency to follow it where the water ratio and cement content are properly balanced with the means used for placing.

It is interesting to note that 1.25 bbl. of cement with a water ratio of 0.60 by weight, and a cement content of 1.50 bbl. with a watar ratio of 0.50 by weight were necessary with the flat coarse aggregate before the curve followed the water cement ratio theory. The more workable concrete provided by the cubical coarse aggregate reacted more quickly and indicated its agreement with 0.75 bbl. of cement

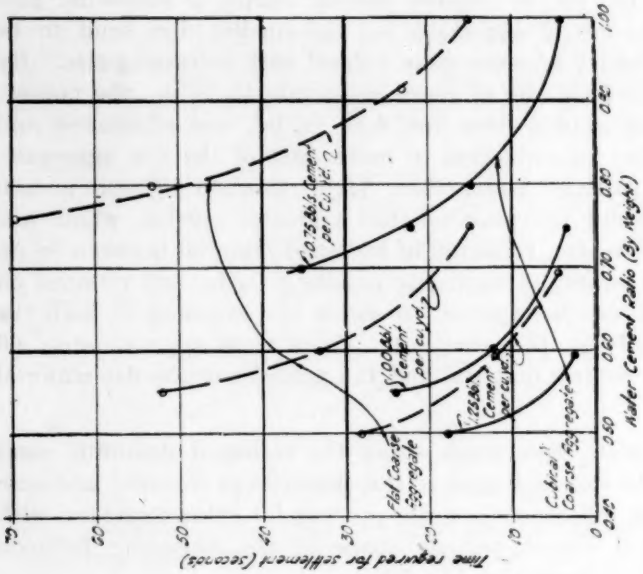

Time required for settlement (seconds)
Water Cement Ratio (By Weight)
Flat Coarse Aggregate
Cubical Coarse Aggregate
1.25 Bbls Cement per Cu. Yd.
1.00 Bbl Cement per Cu. Yd.

FIG. 13

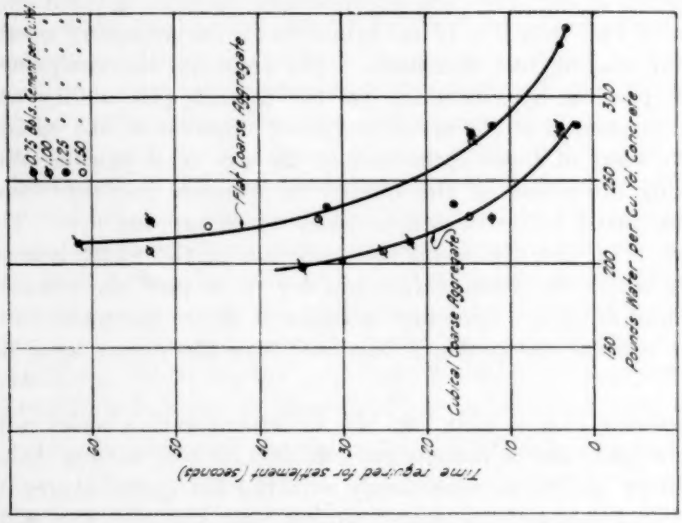

Time required for settlement (seconds)
Pounds Water per Cu. Yd. Concrete
Flat Coarse Aggregate
Cubical Coarse Aggregate
0.75 Bbls Cement per Cu. Yd.
1.00 " " " " "
1.25 " " " " "
1.50 " " " " "

FIG. 12

(SEE PREVIOUS PAGE)

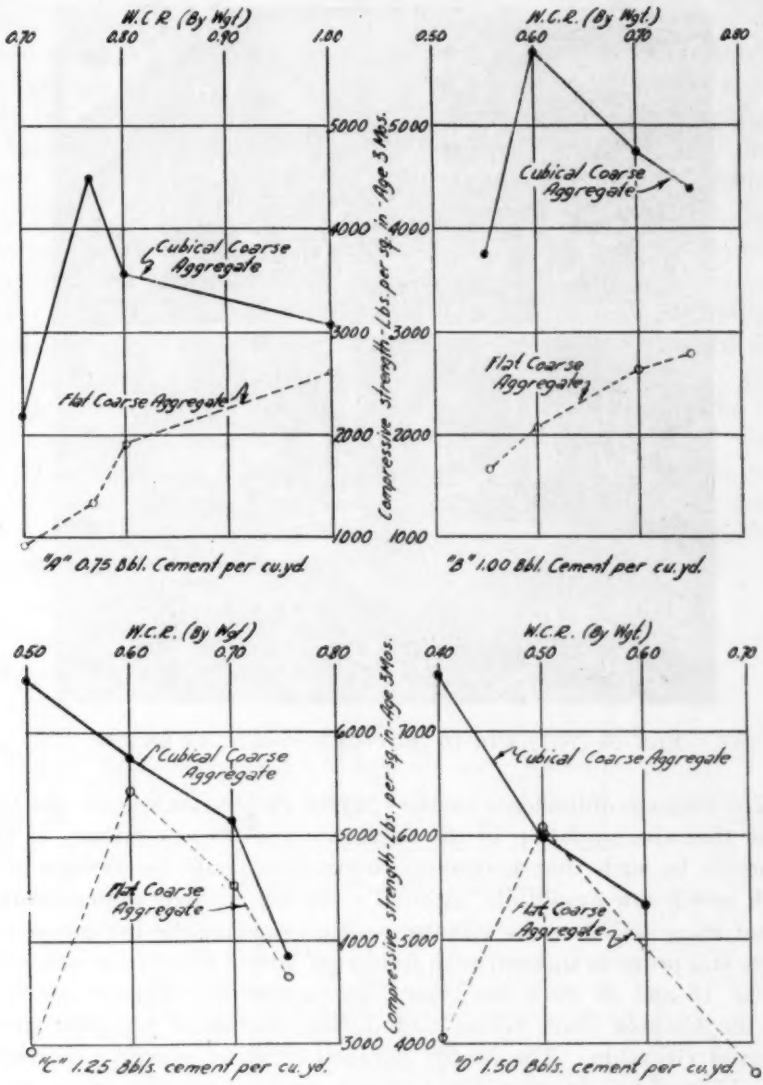


FIG. 14—(SEE TEXT UNDER "2. COARSE AGGREGATE PARTICLE SHAPE")

with a water ratio of 0.80, and a cement content of 1.00 bbl. with a water ratio of 0.60. The significance of this study, when applied to field concrete, is that immediate correction would be required when the concrete failed to settle and compact with relative freedom under vibratory action.

FIG. 15 (TOP) AND 16 (BOTTOM)—SEE TEXT BELOW

The major requirements on the Claytor Project, as regards placing, were that the gradation of the aggregate and the consistency of the concrete be such that maximum compaction could be secured with high speed, low amplitude, internal vibration without segregation or water gain, and that the vibratory action start at the bottom of the mass and progress upward with minimum lateral flow of the concrete.

Fig. 15 and 16 show the typical appearance of the mass concrete on the Claytor Dam before and during placement by high speed internal vibration. The freshly dumped batch of concrete shown in Fig. 15 might indicate a dry or harsh working mixture when compared with ordinary consistencies. In Fig. 16, however, it will be noted that, under the initial vibratory action, sufficient matrix is being brought to the surface to indicate efficient compaction. The controlling factor was one of segregation which limited the amount of the larger size of coarse aggregate. Even with such limitation, enough was present so that the initial vibratory action tended to cause a portion of this larger aggregate to roll down the sides of the

batch and provide evidence of segregation. This loose aggregate was picked up by shovels or stone forks and spread over the top of the batch so that, when the vibratory action was completed, all signs of segregation were removed and matrix was in evidence over the entire area. The more powerful vibrators provided normal compaction. A lighter unit was used specifically for compaction along all lines where the freshly mixed concrete joined the previously placed batches.

3. *Tests to Determine the Effect of Minus* 100 *Dolomitic Aggregate.* The use of the superfine aggregate, or that portion of the crushed stone passing a No. 100 sieve, is an important consideration in concretes of drier consistencies and lower cement contents. Preliminary studies indicated an increase in workability through the use of this material as well as a means through which to control bleeding. Preliminary test results are given in Fig. 17. Chart A indicates the effect of various amounts of minus 100 dolomitic aggregate when tested in mortar by means of 2 x 4 in. cylinders, and Chart B covers tests of concrete in 6 x 12 in. cylinders. Increase in apparent workability due to the addition of this extremely fine aggregate permitted decrease in the water content with accompanying increase in com-

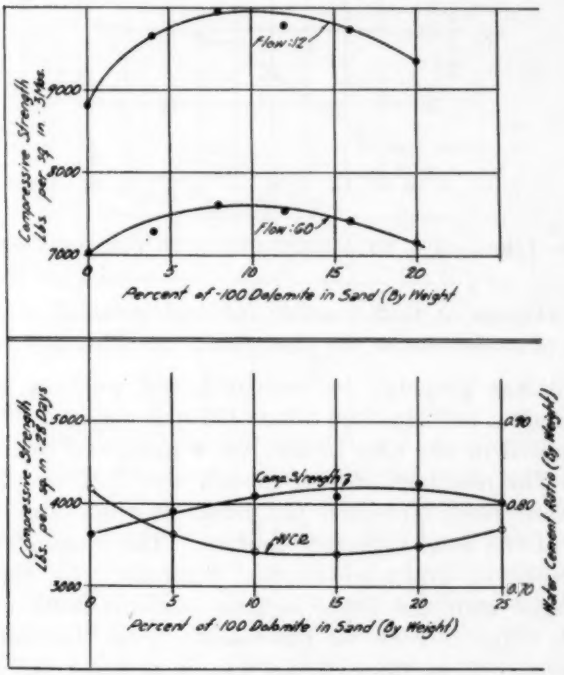


FIG. 17

pressive strength. Further research was carried on to determine the effect of this material when combined with various quantities of cement. These later investigations included two gradations of sand—one size set by the No. 4 sieve, as was used in the dam construction,—and the other having the maximum size decreased so that all sand would pass a No. 8 sieve. The investigation consisted of placeability determinations and compressive tests of 3 x 6 in. test cylinders, with additional specimens for freezing and thawing tests later.

The consistency of the concrete was such that it could be placed properly by thorough hand spading, and would react quickly to high speed internal vibration. It was intended as a consistency to con-

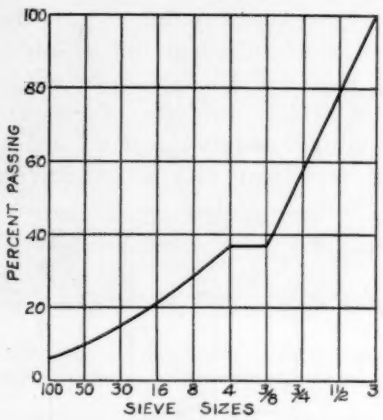


FIG. 18—GRADATION OF AGGREGATE USED FOR MASS CONCRETE

stitute an average of that possible for each method of placing. It had a time of settlement in the placeability machine of 7 seconds.

The sand was prepared by screening and washing so that the resultant product contained no minus 100 material and the maximum size was limited in one case to the No. 4 sieve and to the No. 8 in the other. The resultant fineness moduli were 3.35 and 2.98 respectively. These sands provided the basic mixtures and constituted 50 percent of the total aggregate content. The remainder was composed of $\frac{3}{8}$-$\frac{3}{4}$ in. reshaped crushed dolomite. As expected, the basic mixtures with the lower cement contents were classified as unworkable. The increase in placeability with the water content constant, as the minus 100 material was substituted for the coarser sand in increasing amounts, is indicated by Fig. 19 and 20. Specimens

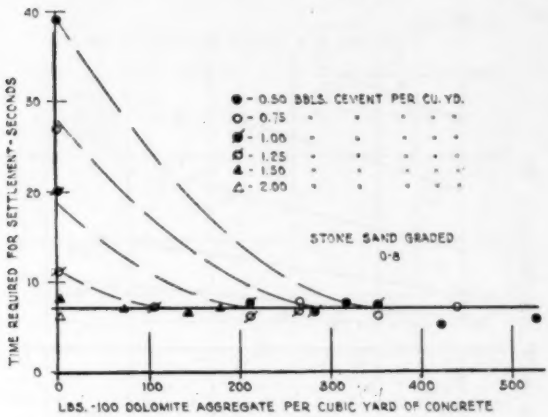


FIG. 19

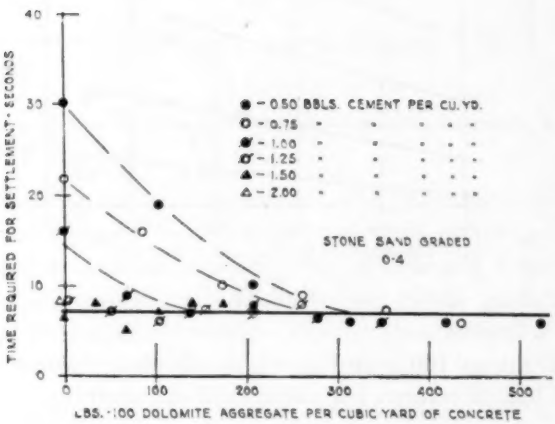


FIG. 20

were made from the mixtures indicated as having equal placeability (settlement in 7 seconds) and also from the basic mixtures, having no minus 100 material present, for compressive tests at the age of 28 days. The results may be noted in Fig. 21. Each point on the curve constitutes the average of five specimens. Certain of the specimens having low cement contents, and no minus 100 material, were considerably honeycombed even though fabricated by vibratory means. The maximum amount of minus 100 material used in these tests, when considered as a paste-making material, provided total solids

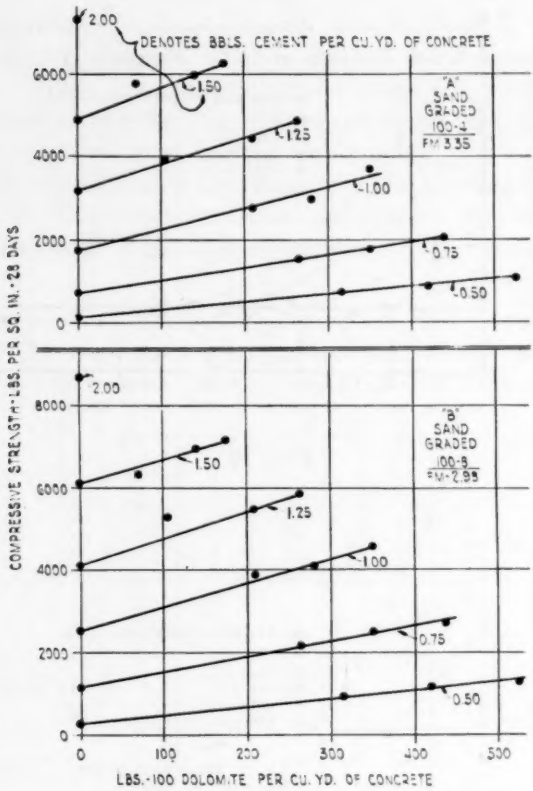


FIG. 21

(cement and minus 100 material) on an absolute volume basis, equal to that of 2 bbl. of cement per cubic yard of concrete.

An interesting point in these investigations was the fact that, to secure equal placeability, the sand graded to No. 8 maximum size required less water than that graded to No. 4. It indicated that the greater gap between the maximum size of the fine aggregate and the minimum size of the coarse aggregate resulted in ultimate compaction of the concrete in a shorter time. The total water content with the 0-8 sand was equal to 272 lb. per cubic yard of concrete with the cement contents up to and including 1.25 bbl. per cu. yd. That for the 0-4 sand was equal to 285 bbl. Slight and consistent increases in water were required in each case as the cement content was increased above this amount. The lower water content in the case of the finer sand is reflected by higher compressive strength.

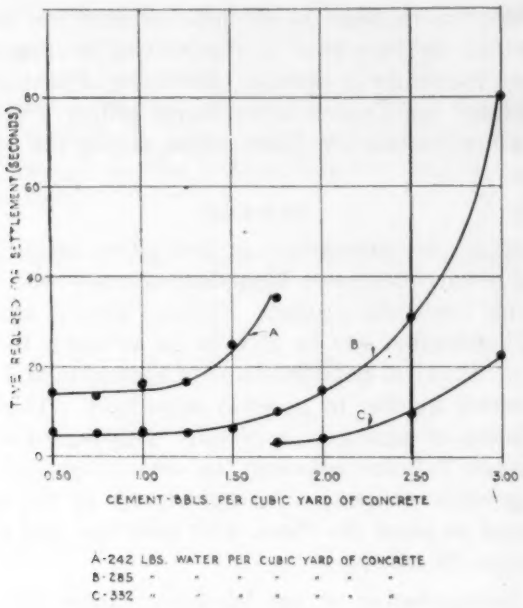


FIG. 22

4. *Effect of Cement on Placeability.* Fig. 22 indicates typical results of an investigation to determine the effect of cement on placeability. It covers a gradation composed of reshaped dolomitic aggregate graded from 0 to ¾ in. Any variation in the gradation or particle shape of the aggregate would influence this particular relation.

General.

While the foregoing has considered the developments more related to aggregate and control of the individual batches up to the time at which they became an integral part of the dam, certain initial practices of a construction nature, were subsequently changed in attempts to enhance the value of the structure as a whole. These included the elimination of longitudinal construction joints in the individual lifts; limitation of the height of lift to 5 ft., with foundation pads immediately adjacent to rock not over 2½ ft.; a minimum of three days' delay before placing new lifts on previously placed concrete; longer periods before form removal; and lower temperature of the concrete during cold weather. Metal water seals, placed adjacent to the upstream face of the dam, were used in all construction and contraction joints.

The combined advantages of all developments can best be judged by the fact that the lake level at this writing is approximately that of the top of the spillway section. Evidence of leakage is missing, except in isolated cases noted in the lower gallery. These consist of seepage at a few contraction joints made during the early period of construction.

SUMMARY

The investigations summarized in this paper are not intended to be of general research nature. They dealt directly with the problems relating to this particular project. Without doubt, some results will find general application and be of value in attempts to improve concrete quality. Study of particle shape of aggregate and the measurement of concrete quality in place is important. The latter would include gradation of aggregate, especially with regard to the maintenance of proper balance between the cement and the minus 100 dolomitic aggregate contents; the consistency of the concrete; and the means used to place the mass, with each one being more or less dependent upon the others.

Practical interpretation of the laboratory tests plus the benefits of field observation resulted:

(1) In the manufacture of approximately 160,000 tons of crushed stone fine aggregate at the dam site which eliminated importing commercial natural sand of relatively inferior quality from a distance of more than 200 miles, at about three times the cost.

(2) In the development of a crushed stone concrete aggregate, both fine and coarse, to the extent that, through changes in particle shape and gradation, etc., an improvement in placeability resulted and permitted a decrease of nearly 0.50 bbl. of cement per cubic yard in approximately 160,000 cu. yd. of concrete. This saving in cement was made with no sacrifice of compressive strength, but with considerable increase in such qualities as directly affect volume changes.

Attempt has been made to reduce the developments which led to lower construction costs of the Claytor Hydro Project to simplified reading. It does not follow that the details involved in attacking the problems were of equally simplified nature. Selection of the most efficient particle shape and gradation of aggregate, etc., did not mean that production was a matter of course. Without the complete cooperation of all concerned, combined with study and experiment by those responsible for the production of the aggregate, the increased quality of concrete and subsequent reduction in cost would not have been possible.

The Claytor Development was constructed by Rinehart & Dennis Co., Inc., General Contractors, Charlottesville, Va. The engineering and supervision were performed by American Gas & Electric Service Corporation, New York City, Philip Sporn, Chief Engineer. F. W. Scheidenhelm, New York City, was the Consulting Engineer, and Roy W. Carlson of Boston, Massachusetts, served as consulting engineer on concrete.

Discussion, to close June, 1940 JOURNAL, should reach A. C. I. Secretary in triplicate by Apr. 1, 1940.

Vol. 36 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE
(Supplement)

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN *SEPTEMBER 1940*

Discussion of a paper by G. W. Hutchinson:

Concrete Aggregate Development on the Claytor Hydro Project*

BY C. A. G. WEYMOUTH†

This paper furnishes a welcome confirmation of the practice that has been developing on the Pacific Coast for more than ten years, that of adding fine blending sand to concrete to improve the quality as well as the workability of the mix. Today local specifications provide for its use, and contractors eagerly desire it when the sands are washed too thoroughly.

In the Los Angeles district, most sands are sharp and angular but not so harsh as the manufactured stone sand described in the paper. Our fine blending sand is a natural product obtained from the washings of the concrete sands by means of water classification. It ranges from the No. 200 to the No. 48 sieves with some overlapping of coarser sizes. The amount needed for the blend depends upon the consistency of the mix; for ordinary building concretes this is the quantity that will increase the amount passing No. 48 sieve to about 20 per cent of 0 - 4 sand. In the sand used in the Claytor mass concrete, 27 per cent is finer than the No. 48 and 16 per cent passes the No. 100 sieve. This agrees well with the writer's experience in correcting local sand adulterated by too much rock screenings. Rock dust has been tried locally in place of natural fines but does not work so well. The writer in a recent paper‡, has given examples of fine and coarse sands with the principles governing blending.

*JOURNAL, Amer. Concrete Inst., Jan. 1940, *Proceedings* Vol. 36, p. 373.
†Concrete Engineer, Raymond G. Osborne Laboratories, Los Angeles.
‡C. A. G. Weymouth, "A Study of Fine Aggregate in Freshly Mixed Mortars and Concretes," *Proceedings*, A.S.T.M., Vol. 38, Part II, p. 354, 1938.

Influence of Gap Gradings

Of his series of concretes with two sands, one 0 - 8 sieve and another 0—4 sieve, and an equal amount of ⅜ - ¾ in. reshaped dolomite, the author states: "An interesting point in these investigations was the fact that, to secure equal placeability, the sand graded to No. 8 maximum size required less water than that graded to No. 4. It indicated that the greater gap between the maximum size of the fine aggregate and the minimum size of the coarse aggregate resulted in ultimate compaction of the concrete in a shorter time."

The author provides no explanation of this observation. A proper understanding of this condition is extremely important, however, since two questions may be suggested: "Are gap gradings better for concrete in their own right than continuous gradings?", and "Are 0 - 8 sands better than 0 - 4 sands for ordinary concrete mixtures?" The answer to each question is, of course, "No." The phenomenon of the fine sand using less water than the coarse sand is due to particle interference between the coarse aggregate and the 8 - 4 sizes of the 0 - 4 sand, while no particle interference by the coarse aggregate is present in the concretes with the fine sand. The increase of voids in the 0 - 4 concretes by particle interference requires a similar increase of the water to give equal placeability.

Particle interference occurs if the magnitude of the clear distance between the particles of the coarse group is smaller than the next smaller group size in the grading. This clear distance depends upon the concentration, in the mix, of the large particles. It may be computed by the writer's formula*:

*C. A. G. Waymouth, "Effects of Particle Interference in Mortars and Concretes," *Rock Products*, Feb. 25, 1933, p. 26.

T. C. Powers, "A Discussion of C. A. G. Weymouth's Theory of Particle Interference," Papers of the Portland Cement Assn., Chicago, March, 1936.

M. F. MacNaughton, "The Structure of Concrete Mixtures," *The Engineering Journal*, Aug., 1933.

C. A. G. Weymouth, "Designing Workable Concrete," *Eng.-News-Record* Dec. 29, 1938.

This being the first description of particle interference in the JOURNAL (See "The Application of Some o fthe Newer Concepts to the Design of Concrete Mixes" by W. M. Dunagan, A. C. I. JOURNAL, June, 1940) a brief explanation of the terms in this formula is given:

Density is used as the complement of voids. The concentration of the particles of a size-group is measured by the ratio of the sum of their volumes to that part of the concrete volume related to these particles. The largest particles of aggregate can alone be spaced throughout the concrete volume. The next smaller particles can be distributed only in that volume of the mix outside the boundaries of the particles of the first group. In general the volume related to any size-group is the total volume of the concrete less the sum of the volumes of all the particles of larger size. The concentration of a size-group is called its relative density, d_a. The concentration of this same group when dry-rodded is denoted by d_o. These densities are expressed as decimals of a unit volume of mix.

The ratio, d_o/d_a, is proportional to the dilation of the particles of a size-group from the dry-rodded condition to the dispersed condition of these particles in the mix. The range of sizes in a size-group varies from the diameter of the smallest particle to that of twice this diameter. When dry-rodded these various sizes constituting the group are in random contact. When dilated in uniformly mixed concrete, the distances between the centers of the particles are increased in the same proportion in all directions. As their separation increases, d_a becomes smaller and the ratio, d_o/d_a, becomes larger. The density, d_o, depends upon the shape and surface texture of the particles and is independent of the diameter of the size-group; it varies from 0.650 for smooth spheres to 0.550 for flat shapes. For the reshaped cubical dolomite in the investigation by the author, d_o is about 0.570. Relative density is determined by first computing the absolute volumes of the size-groups in terms of a unit volume of concrete, the amount for each being found by its proportion in the combined grading; the volume related to a size-group is next computed as outlined above; d_a is then the ratio of the absolute volume of the group to its relative volume in the mix.

$$t = \left[\left(\frac{d_o}{d_a}\right)^{1/3} - 1\right]D$$

in which the diameter D for the $\frac{3}{8}$ - $\frac{3}{4}$ in. group, is 0.556 in., and its dry-rodded density d_o can be assumed as 0.570. The relative density, d_a, varied from 0.363 to 0.389, and the clear distance, t, varies from 0.076 yo 0.093 inches as is shown in Table 1.

Upon comparing these values of t with the diameters of the top size-groups in the sands (D_1 = 0.141 in. for the 8 - 4 group in the coarse sand and D_2 = 0.071 in. for the 14 - 8 group in the fine sand), it will be seen that t is greater than D_2 but much smaller than D_1. There is sufficient clearance between the $\frac{3}{8}$ - $\frac{3}{4}$ in. pieces to permit the D_2-particles to become uniformly distributed in the concretes with the fine sand. In the concretes using the 0 - 4 sand, on the other hand, this clearance is much too small to permit free movement of the D_1-particles. This particle interference results in partial or complete segregation of the two conflicting groups at various points in the mass of the mix; new voids are created thereby, and the mixing water must be increased to fill them to maintain the consistency. The author's experiments illustrate particle-interference effects nicely.

TABLE 1—PARTICLE INTERFERENCE CAUSED BY EXCESS OF $\frac{3}{8}$-$\frac{3}{4}$ IN. GROUP

Sand in concrete	0 - 8 Sieve		0 - 4 Sieve	
Gap in grading	Two Groups No. 8 to No. 4 No. 4 to $\frac{3}{8}$ in.		One Group No. 4 to $\frac{3}{8}$ in.	
Concrete data				
Cement per yard, bbl	0.50 to 1.25		0.50 to 1.25	
Mixing water, lb	272	272	285	285
Composition of concretes by absolute volumes per cu ft. of mixes				
Cement	0.036	0.089	0.036	0.089
Water	0.161	0.161	0.169	0.169
Air, est. at 15% of water	0.024	0.024	0.025	0.025
Sand	0.390	0.363	0.385	0.359
$\frac{3}{8}$—$\frac{3}{4}$ in.	0.389	0.363	0.385	0.358
	1.000	1.000	1.000	1.000
Particle interference				
Assume d_o for $\frac{3}{8}$—$\frac{3}{4}$ in. group is 0.570. D = 0.556 in.				
Top group in sand	No. 14 to No. 8		No. 8 to No. 4	
Diameter, in.	D_2 = 0.071		D_1 = 0.141	
d_a ($\frac{3}{8}$—$\frac{3}{4}$ in.)	0.389	0.363	0.385	0.358
Clear distance, t, in.	0.076	0.090	0.078	0.093
t/D_2 and t/D_1	1.07	1.27	0.55	0.66
Particle interference	none	none	extreme	extreme

Vol. 36 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN *JANUARY 1940*

Report of American Concrete Institute Committee 711

Precast Joist Concrete Floor Systems*

F. N. MENEFEE, CHAIRMAN

1—PURPOSE AND SCOPE

Committee 711 undertook a critical examination of precast concrete joist and superimposed concrete floor systems and construction methods as practiced in the United States in order to place before the Institute a comprehensive yet brief report of the state of the art along with recommendations which it is believed are sufficiently conservative for general use. The report is presented as information and it is not now recommended that it be adapted as an Institute Standard.

While the precast joist and floor systems are preeminently an application and combination of known ideas to produce a new concrete commodity, there are generally two steps in the procedure that trouble the mind of the conservative architect and engineer; namely:

a. Are the joists as furnished by manufacturers adequate to sustain the design loads satisfactorily according to the accepted principles of reinforced concrete design, and can they be made, subsequently delivered to a job, and erected without structural damage?

b. Can a floor be placed or poured or a precast slab be placed over the top of a joist in a way that the joist and floor will develop T-beam action in accord with the accepted mathematical theory of reinforced concrete design?

It is the aim of this report to bring together the salient facts showing that a correctly designed concrete beam can be satisfactorily made in a machine or in a factory and how the beam so manufactured can be made to develop T-beam action in connection with a floor slab; to

*This report was received by the Institute July 25, 1929. It was submitted to letter ballot of the committee which consists of 10 members of whom 9 voted affirmatively and 1 refrained from voting.

record the design and manufacturing procedure which seem most likely to produce satisfactory results. The Committee recognizes that there are other types of precast floors, in which the precast unit is both joist and floor, but feels that the demand for information on the type herein reported justifies consideration independent of other types, which it is hoped will be the subject of a subsequent report.

2—TYPES OF PRECAST FLOOR SYSTEMS

Floors made of precast joists mortised or embedded into a monolithic floor placed or poured on the job are called *precast joist cast-in-place concrete slab* floors, and are called type 1, in this report.

Floors made of precast joist over which are laid precast slabs for flooring are called *precast joist* and *slab* concrete floors, and are called type 2, in this report.

Type 3 floor system is that in which, for one reason or another, it is assumed that the floor slab, precast or job-placed, does not act monolithically with the joist; or that T-beam action does not take place.

3—GENERAL CONSIDERATIONS

It is recognized that precast joists have been used for some time. E. L. Ransome patented a joist in 1902, which was employed in some buildings, but only in recent years has the precast joist become a significant factor in a commercial sense. From 250 to 300 concrete products manufacturing plants throughout the country are adding equipment for making precast joist and floor slabs and, with an increasing demand for fire-resistant, non-settling, relatively non-shrinking building material, it appears that concrete floor systems have an assured future. For the type under consideration, this prediction is supported by critical analysis of the mechanics involved and by tests which show that such floor systems can and are being made which fall substantially within all the requirements set forth in the "Building Regulations for Reinforced Concrete" of the American Concrete Institute for 1936.

Precast joist and floor systems are following the history of practically all material development, namely, that improvement in manufacturing or hand made methods reduces cost, results in a wider field of use and in many cases produces a superior product. Carpenter made concrete beam forms for light construction, on the job, would be too costly. The machine made beam is easily supervised as to quality and placing of material, both of which are important. Manufacturing conditions from the mixing of the materials to and through

the curing process can be kept uniform. Thus by bringing the refining influence of properly supervised manufacturing methods to bear on the old hand made methods of making concrete beams, the dependable joist is made possible.

4—GENERAL ENGINEERING CONSIDERATIONS AND REQUIREMENTS

a—The precast joist concrete floor is primarily suited for relatively light loads. The logical field for its use is in the market now generally served by wood or light steel joist, particularly in houses, small apartments, schools and churches. Precast joist concrete floors are not recommended where load conditions, such as operating or moving machinery, cause heavy impacts or vibrations. Such conditions call for special study and a possible increase in the design load to twice the static load to be sustained, and usually a heavier type of reinforced concrete floor will prove more suitable and economical.

b1—Computations for determining permissible loads follow the standard formulas for rectangular beams where the slab is independent of the joist and for T-beams where the floor is "cast in place," (type 1), or (type 2) where the floor is precast but mechanically bonded to the joist as by notches in the bottom of the slab and top of the joist, assisted by a mortar bed to smooth out irregularities and to improve the longitudinal shearing resistance of the notches. (See Fig. 1 and 2, for illustrations of the two types.)

According to tests on both the precast joist job-placed concrete floor and the precast joist-precast slab the load at failure will be, for the usual spans of 8-in. joist and 2-in. slab, two and one-half to three times the theoretically computed load, using f_s as 20,000 p.s.i.[1] With a properly constructed bond joint between slab and joist, failure will, with increased loading, usually occur through progressive growth of fine tension cracks which may become visible in the lower portion of the joist at about 50 per cent of the allowable live load. These cracks develop into diagonal tension cracks in the web at live loads of two and one-half to three times the computed or allowable live load. Thus the factor of safety compares favorably with that expected in heavier construction, and applies to the precast joist-precast slab type as well as to the precast joist-job placed floor.

b2—The load deflection curve up to the elastic limit of the steel approximates a straight line and at the computed allowable live load the deflection at mid span is about one-third of the usually accepted maximum of 1/360 of the span. This applies to both the job-placed and precast floors.

[1]JOURNAL Amer. Concrete Institute, March-April, 1934, Table 4, p. 321. Load Capacity of Precast Joist Floors, R. E. Copeland and P. M. Woodworth.

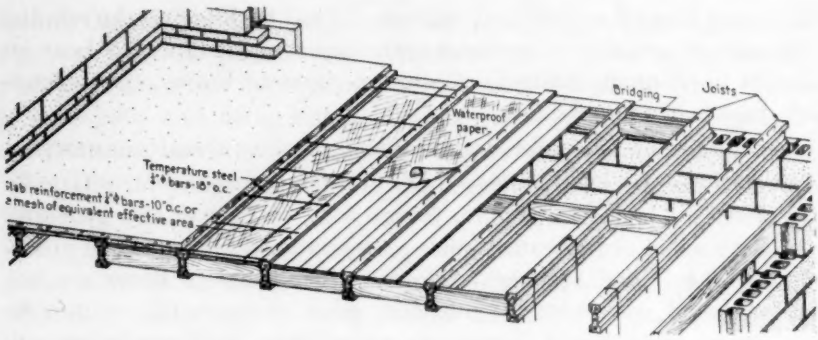
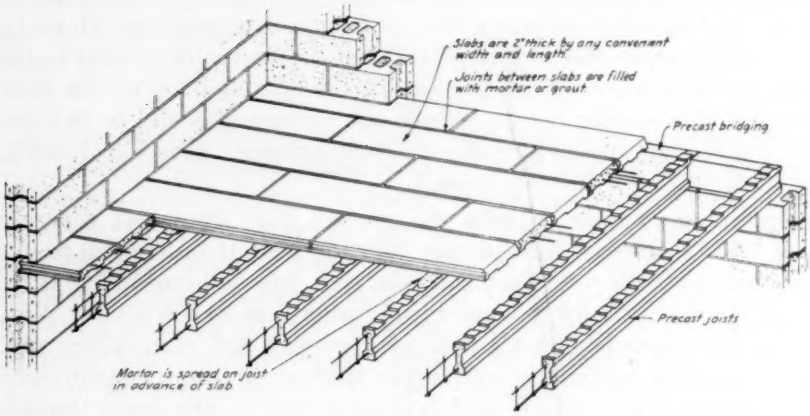


FIG. 1—TYPE 1 FLOOR—PRECAST JOIST, JOB-PLACED SLAB


Slabs are 2" thick by any convenient width and length
Joints between slabs are filled with mortar or grout
Precast bridging
Precast joists
Mortar is spread on joist in advance of slab

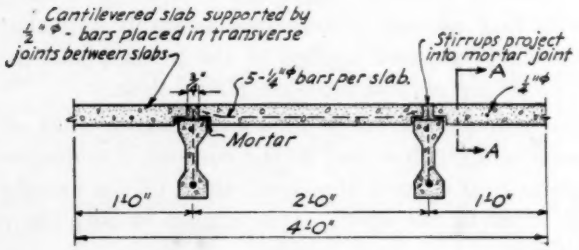


FIG. 2 (a-ABOVE)—TYPE 2 FLOOR (b-BELOW)—DETAIL OF BOND JOINT WITH SLABS PARALLEL TO JOIST

b3—From tests conducted to date there should be no question about the static structural rigidity of the joint between the floor and the joist for either type of construction. In the job-placed floor with the joist imbedded in the under side, the top of the joist cannot move appreciably laterally and in the recommended precast slab type it does not. Tests on spans up to 20 feet do not indicate a need for bridging to prevent side sway of the bottom of the joist. There is a need for tests to show the amount of bridging needed for joists used as independent beams particularly where the floor slab does not provide lateral restraint.

The question of the bond between the joist and slab has been the subject of considerable study and experimentation in test panels and specially designed bond test specimens.[1] The results seem to show that the precast joist and job-placed floor will bond together with longitudinal shear stress resistances amounting to five or six times that induced by the allowable loading and that the factor of safety against failure by diagonal tension is from two and one-half to three times that induced by the allowable shear stress.

b4—While there are no published reports on the long time behavior of these types of floor in normal service, type 1 floors have been in use for ten years in Kalamazoo, Mich. Recorded tests seem to indicate that except for a possible slight modification due to plastic flow the physical performance should be permanent. The compressive stresses are in the concrete floor slab as in all concrete floor construction, the reinforcing taking the tensile stresses as a truss encased in some form of concrete, which assists in making it stable against side sway. As long as the bond between the slab and joist remains unimpaired, T-beam action will result. Since the major portion of the joist shrinkage occurs before being placed in the floor, stresses set up at the junction of the slab and joist from shrinkage should be less than in monolithic construction. From our experience with metals we know that secondary stresses are set up at re-entrant angles where non-uniform shrinkage takes place, and reasoning by analogy there are probably similar stresses in similar shapes in monolithic concrete.

b5—Excess loading, if sufficient, will cause permanent deflection, In the case of precast slab with no steel through or across the joint, unsymmetrical loading, impact or vibration or large volume changes may in time loosen the bond and thus impair the T-beam action. Tests for these conditions should be arranged.

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[1] "Concrete Slabs and Precast Joists Act as Monolithic T-Beams," F. N. Menefee, *Concrete*, March, 1934.

"Some Tests of Load Capacity of Floors Made with Precast Concrete Joists," R. E. Copeland and P. M. Woodworth, JOURNAL Amer. Concrete Inst. Mar-Apr. 1934, *Proceedings* Vol. 30.

b6—With the job placed concrete floor slab, the transverse rigidity of the floor may be expected to be equal to that of any concrete floor of like thickness and on joist of the same spacing. Precast slabs may not give the same results, yet where they are properly keyed together and grouted along their sides and ends, there will probably be little if any difference in transverse rigidity, and the same distribution of load will probably result. Tests to confirm this prediction should be arranged.

b7—The coverage of the steel is slightly under the requirements called for in the standard specifications.

b8—The job placed floor slab with or without the shear loops protruding above the joist seem to be effectively anchored to the joist. In the reported tests on precast slabs under concentrated loads, uplift of the slab at points not under the load occurred only after failure in diagonal tension had reached the top of the joist. For static loads under normal conditions of moisture and temperature, the notched and mortared joint between the slab and joist appears to provide sufficient anchorage.

5—GENERAL MANUFACTURING AND ERECTION CONSIDERATIONS

Installation of precast-joist-making machinery by experienced concrete masonry manufacturers throughout the country has been going on for five years or more. Their experience in making concrete masonry contributes materially to the successful utilization of the idea. The management and the plant personnel are already familiar with the many details of mixtures, curing and handling of concrete products. They understand the necessity for close supervision and careful attention to the minute details and of conforming to the requirements of the proper physical and time relationship between cement, aggregate, design, and a finished product that will meet specifications. A survey of the industry indicates that they have developed special machines for making the joist or that they use steel forms and vibrating equipment in order to obtain the necessary control and uniform quality of their product.

Manufacturers from various parts of eastern United States have submitted estimates on probably maximum economic haul varying from 50 to 150 miles. A Kansas City, Missouri, manufacturer advised the Committee that "delivery by truck within a radius of 150 miles does not add more than 15 per cent to the quoted price." Another says "additional trucking costs add but little to the square foot cost of the floor." However, a third says that "75 miles is the maximum

if there is no other factory in the territory, otherwise a 50 mile radius is the limit because hauling costs become excessive."

Damage to joist in the factory and between the factory and the job is uniformly reported as none or negligible. "It has been necessary in a few cases to point up small imperfections, which is easily and quickly done."

One manufacturer reports that, "since 1934 when we began making precast joist we have had less than five broken joists in more than 90,000 lineal feet delivered." These statements indicate that it is both practicable and economical to make precast joists in a permanent plant and to transport them considerable distances to the job.

Upon arrival at the job, the joist is manually set and properly bridged at the supports, by brick, concrete block, short lengths of joist, or concrete. All this work is done by masons and their helpers although there is no reason why any reasonably skilled workman should not be able to lay such a floor, provided he follows instructions from an architect or as outlined in pamphlets issued by the Portland Cement Association.

As long as the joist is not too heavy to handle by four men or in some cases by light derricks, it appears that the precast joist will be more economical and a better joist than that one which is cast in place. The economical and safe limits of largeness for precast joist and smallness for cast-in-place beams and girders has not been established, although at the present time precast joist sizes do not exceed a depth of 12 in., 24 ft. long. Such a joist weighs approximately 650 lb. if made of light weight aggregate and as much as 840 lb. if of ordinary sand and gravel—they fall within or approach the weight where erection by machinery is necessary.

As in most matters of an engineering nature, we can be certain at extremes and somewhere between the extremes we should be hesitant; accordingly there is a zone in which the precast joist is unquestionably superior from every standpoint, and another zone in which it would be impracticable and certainly uneconomical and still another zone in between, where the proper choice is a matter of opinion.

6—TESTING PRECAST FLOOR SYSTEMS AND UNITS

As already shown, the types of precast joist concrete slab floors now most commonly employed have been subjected to sufficient testing to demonstrate their load behavior and structural properties. However, the committee recommends that joist manufacturers have tests made periodically on concrete cylinders to provide assurance that the joist concrete has a minimum compressive strength of at least 3,000 p.s.i.

When developing new systems or changes from the joist or construction details shown herein the manufacturer should test a number of full size panels approximately 4 ft. wide by 14 ft. long. Preferably such tests should be supervised by a competent engineer. In the public's interest it is obvious that new systems should not be marketed until their structural properties have been investigated and proved adequate.

If a load test is required or seems advisable in the case of a floor already installed in a house or building it should be performed as prescribed in the American Concrete Institute Building Code: "Building Regulations for Reinforced Concrete, Section 202—Load Tests.

7—RECOMMENDED PRACTICE IN THE DESIGN AND CONSTRUCTION OF PRECAST JOIST FLOORS

Type 1 floor is made up as shown in Fig. 1. First the precast joists are set at the predetermined spacing; after which braces and spreaders are placed. Form material upon which to place the job-placed concrete floor is then laid on top of the spreaders, in such a way as to produce an imbedment of the head of the joist into the under side of the floor from ½ to ¾ of an inch. This imbedment alone will insure T-beam action from the bond of the slab concrete to that of the joist, sufficient to resist failure at the joint, but with the shear loop from the joist firmly imbedded in the floor concrete, the ultimate shearing resistance longitudinal of the joist is increased.

Type 2a floor is made of precast joists transversely notched on top and reinforced with the same size steel top and bottom as the joist in type 1. The shear rods in type 2 may or may not protrude above the top of the beam, depending upon the way in which the floor slabs are made. In Fig. 2a the floor slabs are laid transversely of the joist and are notched on the under side to match the notches in the top of the joist or to receive a mortar key that is similarly keyed into the joist. In this type, which will be referred to as type 2a, there are no shear loops protruding from the top of the joist.

In type 2b the precast floor slab is laid longitudinally of the joist with enough clearance between the slabs along the top of each joist to permit the use of a joist with protruding shear loops. Fig. 2b[1] shows a method of moulding end rabbets in each slab to increase the stability of the joint.

Type 1 floor seems now to be more popular with the architects or builders probably because its structural stability is more apparent and because it gives an impression of being more enduring in service.

[1]Copeland and Woodworth, JOURNAL, Amer. Concrete Inst., Nov.-Dec., 1935, *Proceedings* Vol. 31

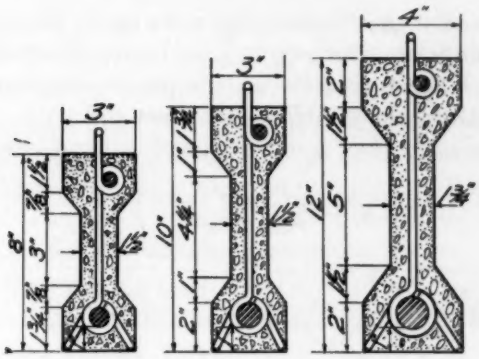

FIG. 3—JOIST CROSS-SECTIONS

	8" Joist		10" Joist		12" Joist	
Sectional area	18.2 sq. in.		22.1 sq. in.		33.4 sq. in.	
Aggregate	Lightweight	Ordinary Weight	Lightweight	Ordinary Weight	Lightweight	Ordinary Weight
Ave. wt. of joist, lb. per lin. ft.	14	19	17	23	26	35
Maximum span	16 Feet		20 Feet		24 Feet	

The assumptions concerning variation of stress, and of elasticity as applied to reinforced concrete in standard treatises, and as recommended by the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, in its *Progress Report*, January, 1937, is so inclusive and well established that changes in fundamental assumptions or mechanics of design are not to be expected. Accordingly the Committee sees no need for the introduction of assumptions or mechanics not approved by the above mentioned specifications.

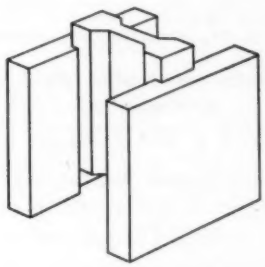

FIG. 4

For types 1 and 2 the fundamental question is the matter of bond between slab and joist. Both types have been subjected to the test of service and to laboratory tests on specimens designed to show the shearing strength of the joint between joist and slab. The bond has been found adequate. Fig. 4 shows a bond test specimen representing

TABLE 1—RESULTS OF MORTISED JOINT BOND TESTS WITH NO REINFORCING ACROSS THE JOINTS*

Specimen	Age Days	Mix	Bond Resistance at Failure, p.s.i.	Comp. Strength p.s.i.
Test of January, 1932				
1	16	1:2:3	432	
2	13		528	
3	28	1:2:4	482	1860
4	28		312	1990
5	28		460	1990
6	28		535	1700
7[1]	34	1:2½:3	606	3270
8	34		574	
9	58		650	3060
10	58		587	
11[2]	38	1:2:4	346	685
12	38		269	730
13	38		305	
14[3]	38		370	
Tests of January, 1933				
1	21	1:1.5:4	360	2175
2	31	"	327	2330
3	31	"	310	2450
4	31	"	180	1600
5	31	"	295	2170
6	31	"	301	2110
Tests of February, 1933				
1	31	1:2:4	276	2700
2	31	"	196	
3	31	"	562	4310
4	31	"	522	3790
5	31	"	338	2870
6	31	"	355	3410
Tests of March, 1933				
1	28	1:2:3	530	
2	28		501	
3	28	1:2.15:2.85	450	
4	28	"	210	
5	28	"	200	
6	28	"	130	
7†			340	

[1]. Failure by shear in material of joist.
[2]. 2 per cent calcium chloride by weight of cement.
[3]. 4 in. silo stave replacing precast joist.

*By F. N. Menefee, *Concrete*, March, 1934.
†By Copeland and Woodworth, JOURNAL, Amer. Concrete Institute, Mar.-Apr., 1934.

TABLE 2—RESULTS OF MORTISED JOINT BOND TESTS WITH REINFORCING ACROSS JOINT*

Specimen	Joint Resistance at Failure, p.s.i.
1	280
2	300
3	370
4	300
5	310
6	450

*Copeland and Woodworth, JOURNAL, Amer. Concrete Institute, Mar.-Apr., 1934.

type 1, and Table 1 shows the results of several series of tests with no reinforcing across the joint; while Table 2 shows the results of bond resistance with various types of shear reinforcing across the joint.

On 16 panels made up of two joists spaced 2 ft.-0 in. center to center, each with 14 ft. span and a 4 ft.-0 in. wide slab, 2 in. thick, Copeland and Woodworth measured a maximum deflection of 0.209 in. at 80 lb. per sq. ft. load. This is less than one-third of the usual 1/360 of the span permitted. The design load, or that which theoretically produced 20,000 p.s.i. tension in the steel, was 85 lb. per sq. ft.

On 13 of the panels tested to failure the ratio of the ultimate load to the design load of 85 lb. per sq. ft. averaged 3.3.

The measured stress in the steel at 160 lb. per sq. ft. of load was about 82 per cent of the theoretically computed stress of 30,650 p.s.i.

While the longitudinal shearing force in the joist at the plane of the bottom of the slab is as great as at any place between the slab and the moment steel, the breadth of the joist at this plane is twice as great as in the web (or more in the 12 in. joist) and hence the shear stress is only one-half as great . The bond area around the joint is greater than the shear area directly across the joist, hence the longitudinal bond stress in type 1 floor is still less than the longitudinal shear stress. These facts, along with the actual panel tests above cited, lead the Committee to believe that failure in type 1 floor will come in some other place than in the bond between the joist and slab.

Method of Computing Live Load Based on Resisting Moment of Steel—Type 1[1]

Assume a floor with 8 in. joist, $\frac{3}{8}$ in. and $\frac{7}{8}$ in. round bars at top and bottom respectively. Spacing of joist 20 in., top imbedded $\frac{1}{2}$ in. into under side of a 2 in. floor slab on a 15 foot span. $f_c = 3000$ p.s.i.; $f_s = 20{,}000$ p.s.i.; $n = 10$; $M = f_s a_s jd$.

If the joist is not supported from below except at its ends, it acts as a doubly reinforced beam and must carry not only its own, but the weight of the plastic floor slab until the slab hardens and T-beam action starts. Under the condition stated we compute the stress in the tension steel from the dead load, then assume the remaining allowable stress is available for the live load and compute the permissible live load by the regular T-beam formulae.

Assuming the joist takes all the dead load before the slab hardens, and allowing an increase in flange width of $\frac{6}{10}$ in. for the $\frac{3}{8}$ in. upper bar in the joist; j will be .83, and d will be 6.56 in. The weight of the joist and floor will be 34 lb. per sq. ft., and $M = 19{,}200$ in. lb. from

[1]Using nomenclature conforming to joint committee recommendations.

which f_s = 5880 p.s.i. This leaves 14,120 to be used in T-beam action for the live load. For the T-beam section j = .92 and d = 8.06 in. We then have $f_s a_s jd$ = 14,120 x .6 x .92 x 8.06 = 62,800 in. lb. or 5220 ft. lb. of resisting moment available for live load, which will be 112 lb. per sq. ft.

Method of Computing Live Load Based on Shear

Assume the same floor as above. Since the floor and joist do not act as a unit until after the concrete in the floor slab has set, it is proper to consider the joist as taking all of the shear of the dead load due to the weight of the joist and slab.

Dead load per linear foot = 57 lb. Maximum shear V = 427 lb., j = .83, d = 6.56 in., b in the web of the joist = 1½ in.

$$v = \frac{427}{1\frac{1}{2} \text{ x } .83 \text{ x } 6.56} = 52.3 \text{ p.s.i.}$$

From Table 8, p. 44, of the Progress Report of the Joint Committee for January, 1937, the allowable shear stress v_c in the concrete is = 0.06 f_c' = 0.06 x 3000 = 180 p.s.i. Thus over and above the dead load there is 180 − 52.3 = 127.7 p.s.i. of shear resistance available to support the live load.

Upon the hardening of the slab, the joist and slab function as a T-beam and j = .92 and d = 8.06 in. Then $V = vbjd$ = 127.7 x 1½ x .92 x 8.06 = 1420 lb. With 20 in. spacing the live load per sq. ft. based on shear =

$$\frac{1420}{1\frac{2}{3} \text{ x } 7\frac{1}{2}} = 113.5 \text{ lb.}$$

The structural bond between the joist and job-placed slab has been found ample on joist up to 4 in. wide. The strong grip of the slab is due to the fact that the joist has had its largest shrinkage and the shrinkage of the slab undoubtedly produces a closer bond than were the joist shrinking along with the slab. In fact, from what we know of stresses in re-entrant angles, it is entirely possible that there are initial shrinkage stresses in the monolithic T-beam that are not at all present in the precast joist job-placed T.

In case the floor joists are amply supported from below, so that little or no beam action is called into play until the floor hardens and T-beam action results, the total load which the beam will support is computed at once by the T-beam formulae.

Type 2 Floors

Ten type 2 floor panels 14 ft. long, consisting of 2-in. floor slab, 8-in. joist on 24-in. centers, and one 20 ft. span with 2-in. slab and 10-in.

joist on 24-in. centers, were tested by Copeland.[1] Out of nine panels with notched beams and transverse floor planks (Fig. 2a), and two panels with shear loops projecting above the joist and into a clearance between longitudinal floor plank (Fig. 2b), only 2 failed in longitudinal shear or bond at the joint between joist and slab. The live loads at which these two failed in bond were 244 and 218 lb. per sq. ft., or from 2.58 to 2.87 times the computed allowable live load. In all cases, including the two bond failures cited, the yield point of the steel had been reached.

Thus, as far as structural design is concerned, type 2 floors can be computed in the same manner as type 1; that is, in both cases the joint between the joist and slab is as strong or stronger structurally as other parts of the beam. Aside from the fact that the joist is assumed to take all of the dead load until the job-placed floor sets, or the mortar under the precast slab hardens, after which T-beam action prevails, the design procedure is the same as in any reinforced concrete beam.

Pending further tests and a longer period for observation of installed precast floor systems, it is recommended that the live load on this type of floor be restricted to 100 lb. per sq. ft.

T-beam action in type 2 floors is more dependent on the quality of workmanship and other factors than where the slab is built in place. It is also possible that vibration and moisture on the under side of the slab may introduce factors relatively unfavorable to type 2.

Type 3

Type 3 precast floor assumes no T-beam action between slab and joist. The joist is considered an independent concrete beam and its shear and bending moment resistance computed exactly as any other doubly reinforced beam is figured. It is recommended that live loads on such floors be not over 100 lb. per sq. ft. and that adequate provision be made against lateral deflection of the joist under load.

Joist Spacing and Floor Thickness

There can be no set rule relative to joist spacing. For a given joist with a given span, a definite resisting moment is available, just as if wood or steel joist were being used. The closer the joist spacing the greater the floor load which can be supported. Single joists are usually spaced from 20 to 33 in. Less than 1½ in. floor thickness over joist may lead to cracking.

[1]Journal Amer. Concrete Ist., Nov.-Dec., 1935 *Proceedings* Vol. 31. One unpublished test of a type 2 panel was made by F. N. Menefee with similar results.

Joist Sections

Joist sections as shown in Fig. 3 have been subject to tests with and without floor slabs over them, as independent and as T-beams. They have been modified where found weak in comparison with other parts and now when properly made, represent a well-balanced beam for resistance to shear and bending moment.

The Committee recommends approval of the dimensions shown; the use of 3000 p.s.i. (minimum) concrete and intermediate grade Billet-Steel Concrete Reinforcement Bars (A.S.T.M. Serial Designation, A15-35) or Rail-Steel Concrete Reinforcement Bars (A.S.T.M. Serial Designation, A16-35) for reinforcement except that rods less than ¼ in diameter shall be cold drawn steel wire (A. S. T. M. Serial Designation, A82-34).

For the upper reinforcing bar a ⅜ in. round rod or its equivalent in area is recommended.

In the 8 in. joist the lower rod will vary from ⅝ in. to ⅞ in. round.

In the 10 in. joist the lower rod will vary from ¾ in. to 1 in. round.

In the 12 in. joist the lower rod will vary from ¾ in. to 1 in. round and in some cases 1 in. square.

Multiple Joist Beams

Multiple joist beams can be used. Their strength will be at least that of the single beam multiplied by the number used. For the architectural effect of multiple joist a wider spacing may be adopted provided the floor thickness is properly taken care of.

Bridging for Lateral Stability

In floor types 1 and 2 stability against buckling is provided by the anchorage between slab and joist.

The independent joist should be laterally braced at midspan between supports when used in spans over 14 ft.

Holes in Web

Holes in the web of the joist reduce the ultimate shearing resistance and should be held to a minimum. Where found to be necessary they should be cast when the beam is made, or drilled on the job (not punched), and be not over 2 in. in diameter. No holes should be made by any mechanic on a job except under the supervision of the architect or engineer.

8—MANUFACTURE

As has been said, the precast joist is not made by the novice. Those becoming interested do so because usually they are in the concrete block or artificial stone business. They are already supplying building

material made of the same ingredients to the trade. They understand the language of the cement and aggregate salesman, as well as that of the contractor.

Therefore the selection of proper aggregate is no great problem to them. A successful concrete block man in a given locality knows about what can be done with the available natural material in his territory and with but little study and experimentation he can make the proper choice between it and shipping in special material.

Some idea of the aggregate now used may be gained from Table 3. It will be noted that the choice of aggregate seems to run to that which is indigenous to the locality. The table does not represent the results of a complete survey of the industry. Its disclosures may or may not be typical. Other aggregates such as cinders and waylite are being used. The committee recommends the use of only those types of aggregates which have been shown by comprehensive tests and experience to conform to the requirements for a high grade concrete.

Joists are stored standing on edge and instructions issued to handlers regarding the proper way to carry them. Where the shear loops do not protrude, the tops of the joist are marked in some other way.

TABLE 3—MANUFACTURING DATA

Location of Manufacturer	Cement	Fines	Coarse Aggregate	Mixture	Water per Sack Cement (gal.)	Admixture	Vibrate	Cure
Kansas City, Missouri	High Early Str.	Kaw River Sand	Crushed Limestone ¼ to ⅝"	1:2.7:3.95 by wt.	6¾	none	yes	air
St. Louis, Missouri	High Early Str.	Meramec River Sand No. 100 to ⅛"	Haydite ¼ to ½"	1:2:3½	5½	none	yes	air but changing to L. P. steam
Des Moines, Iowa	High Early Str.	One part each of finer than No. 6 and finer than $\frac{1}{16}$"	Gravel ¼ to ⅝"	1:2:2	5	none	yes	L. P. steam
Knoxville, Tennessee	Standard Portland	Sand	Gravel and Superock	1:2:2	varies	CaCl	yes	L. P. steam
Evanston, Illinois	High Early Str.	No. 2 Torpedo and B Haydite	No. 6 Gravel and A Haydite	1:1¼: 1½	4	none	yes	air
Detroit, Mich.	High Early Str.	Sand	Haydite Gravel	1:3	6 or 7	none	yes	L. P. steam
Kalamazoo, Mich.	High Early Str.	Sand	Haydite ⅝" to dust	4:10:10	7	none	no	L. P. steam

9—INSTALLATION AND CONSTRUCTION DETAILS

On every job there will be a need for a joist setting plan; prepared by an architect or an engineer and approved by the manufacturer. Only in this way will the owner be assured of unquestionable results. Shears and bending moments will be properly taken into account by the architect or engineer and the manufacturer becomes a party to the transaction. His supervision over the placing of reinforcement and the treatment of the joist from planning the mixture to delivery on the job will have greater attention and advantage if his advice may be had.

There is a need for standardization of many installation and construction details. This does not mean that innovations should be prohibited or frowned upon, but rather that an acceptable practice in handling and setting, leveling, shoring of joists, placing forms for floors, reinforcement for floors, conduits, bulkheads, stairwells, partition bearing joist, etc., should be approved and the architect, engineer, manufacturer and building contractor be informed. The Portland Cement Association pamphlet "How to Design and Build Precast Joist Concrete Floors" is now the best source for information of the sort mentioned.

10—RECOMMENDATIONS

Research in connection with precast floor systems should be continued; some of the as yet unanswered questions could be settled by the following tests:

1. Tests under dynamic conditions such as repeated loadings at 1½ times the design load; this test should be run on floor slabs exactly alike in dimensions but of three types, a. Monolithic, b. Precast joist-cast-in-place concrete floor slabs, c. Precast joist-precast slabs.

2. Tests on comparative distribution of loads as between the job placed floor and the precast slab.

3. Tests or studies as to the desirability of placing a camber in the joist; in both type 1 and type 2 floors.

Discussion, to close in June, 1940 JOURNAL, *should reach A. C. I. Secretary in triplicate by Apr. 1, 1940.*

Vol. 36 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

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Job Problems and Practice

Some will ask questions — Some will answer them — Some will do both

A. C. I. members are invited to use this new JPP department as an informal means toward mutual help. When a problem holds possibilities of general interest the discussion will be briefed in these pages.

If you have a problem—present it; a question, ask it. If you know the answer to a question asked, or if you can contribute something which may help in the solution, or have reason to disagree with the answers or suggestions published, your contribution will be welcome.

If you know of an interesting problem whose solution has already smoothed the way of someone in the field, tell us about it—some other A. C. I. member may need just that information.

The "answers" are the answers of individuals to whom the questions are referred and not of the A. C. I. as an organization.—EDITOR

Sea water for mixing concrete (36-52)

Q—I have a job of concreting to do on a coral island where there is no fresh water. Is it desirable to use sea water to gage the mix?

A—In his paper, "Tests of Impure Waters for Mixing Concrete," (A. C. I. *Proceedings*, Vol. 20, 1924, p. 422) Duff A. Abrams said among his summaries of tests of waters from many sources:

(7) Concrete mixed with sea water (about 3.5 per cent salts, mostly sodium chloride) and cured in the moist room gave higher strengths than fresh-water concrete at ages of 3 and 7 days; at 28 days and over, the strength-ratios for sea water ranged from 80 to 88 per cent. Air-cured concrete mixed with sea water was lower in strength than similar fresh-water concrete at 3 mo., but showed a recovery in strength at later ages and gave strengths equal to that obtained with fresh water. (In spite of the satisfactory strength results, it seems unwise to use sea water in reinforced concrete construction, particularly in the tropics on account of the danger of corrosion of reinforcement.)

(8) Synthetic sea water gave concrete and mortar strengths similar to natural sea water.
(9) Concrete mixed with water from the Great Salt Lake (about 20 per cent sodium chloride) gave strength-ratios from 65 to 77 per cent at ages of 28 days and over. This water is not satisfactory for mixing concrete, unless allowance is made for about 30 per cent reduction in the assumed strength.
(10) Water from Devil's Lake, North Dakota (0.15 per cent sodium sulphate and 0.15 per cent sodium chloride) gave normal concrete strengths and showed no ill effects.
(11) Water from Medicine Lake, South Dakota (3.5 per cent solution of sulphates, largely magnesium; SO_4 concentration 2.8 per cent), gave strengths similar to that obtained with sea water. The lowest strength-ratio was 84 per cent.
(12) Waters from drains and small streams in sulphate districts gave satisfactory strengths at ages up to 2¼ yr. The lowest strength-ratios were about 90 per cent.

From a correspondent we have also these comments and references:

We do not have any references to recently published papers on the use of salt or brackish water used as mixing water in concrete. There are several references in the literature which indicate that salt water has been used in mixing plain concrete without incurring trouble at later periods. This practice is however, discouraged where reinforced concrete is involved. Much of the concrete in the Florida Key structures of the Florida East Coast Railway was mixed with sea water, with no detrimental effect due to its use. Experienced engineers are practically unanimous in the belief that sea water should not be used for mixing in reinforced concrete work, particularly in the tropics. See: "Concrete Viaducts on the Key West Extension of the Florida East Coast Railway," by G. P. Carver; *Eng. Rec.* Oct. 20, 1906; "Bad Effects Resulting from Use of Salt Water in Reinforced Concrete Structures Built in Tropical Countries," by J. L. Harrison; *Abstracts ,Eng. News*, v. 76, p. 1047, Nov. 30, 1916.

A mortar strength test as a measure of the quality of the mixing water is proposed in the Progress Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, Jan. 1937. This specification states, "When subjected to the mortar strength test, the strength at 28 days of mortar specimens made with the water under examination and normal portland cement shall be at least 90 per cent of the strength of similar specimens made with water of known satisfactory quality."*

Out with the "Yard" (36-45)*

Q—The first item in September, "Out with the 'Yard'," seems to merit some comment.

In addition to the two objections mentioned to the elimination of the cubic yard as a concrete measure there would appear to be three others of greater importance.

1. A large body of literature, tables, and other publications exists which would have to be changed or scrapped.

2. Most persons who deal with concrete are accustomed to thinking in terms of the yard.

*See Sept., 1939 JOURNAL, p. 97.

3. There is no sound reason for reforming the relation between two units of measure without putting the entire system on a logical basis.

Most technical publications dealing with concrete, as for example descriptions of actual construction work and laboratory investigations, give data expressed in terms of the yard, most tables showing the relations of quantities of cement and aggregate, etc. to other properties of the concrete are expressed in terms of the cubic yard. Specifications, standards, mixer capacities, mixes, advertising literature, etc., are expressed in terms of the yard. Obviously to translate the first class of publications would be practically impossible and a waste of time. The second and third classes could undoubtedly be changed but would involve an expenditure of effort which might well be put to some more useful purpose.

That most concrete technicians, engineers, contractors and others connected with the use of concrete speak and think in terms of the cubic yard seems fairly well established. To reform these unregenerate individuals seems next to impossible. Probably it would only result in confusion more confounded. This is an objection raised to any attempted reform in nomenclature and is not valid, provided that the reform can be shown to be worth the effort. Such would not appear to be the case with respect to the "yard."

Changing the prices per cubic yard for concrete, one of the objections mentioned, does not seem serious. It involves only a few simple multiplications or divisions and does not seem any more arduous for the supplier of concrete than determining the price of some fraction or multiple of a cubic yard.

The answer to the other objection is most curious. It is suggested that if the units of square and cubic feet are too small, larger units, the "square" (100 sq. ft.) and the "hundred" (100 cu. ft.) should be substituted. (To be logical this last should be the "thousand"—1000 cu. ft.). This really amounts to an injection of the decimal system into our already incredibly illogical system. With this change we would have a relation between inches and feet in the duo decimal system, a relation between feet and the next larger unit in the decimal system and some incredible relation, which the writer has not bothered to work out, between the next larger unit and such units as the mile or the acre. The writer is heartily in favor of changing all our units of measure from the absurd anachronisms which we now use to the decimal system as a whole but realizes the almost unsurmountable obstacles to this ideal. It is not apparent, however, that the substitu-

tion of one relation in the decimal system for one in the monogesimal system offers any advantage or is even a step in the right direction.

In this connection it might be mentioned that in practice the substitution of the "square" (100 sq. ft.) for the square yard seems to have been made already. In some years of experience with floors the writer does not recall ever hearing the area expressed in square yards and seldom has heard of other concrete areas so expressed. General practice with respect to floors is to give the area either in square feet or in "squares" (hundreds of square feet). Floor mixes and tables dealing with materials required for floors are usually expressed as quantities per 100 sq. ft. Coverage of concrete surfaces, whether floors or other types of surface, by paints, curing compounds, etc., are usually expressed simply as square feet. The above remarks, therefore, apply primarily to the cubic yard.

One other question might be asked. Why pick on the cubic yard when we have so many units of measure which do not appear to have any logical reason for existence such as the gallon, the sack of cement, the barrel of cement and many others? A sack of cement is alleged to be a cubic foot but a cubic foot is no more 94 lbs. than it is 100 lbs. or 87½ lbs. or any other figure, within certain limits, which might be selected arbitrarily.

Today there is no particular reason why a barrel of cement should be four sacks of cement or 376 lbs. and the last is certainly a much more awkward relation to handle than that of 27 cu. ft. to 1 cu. yd. There is no more and no less reason to preserve the gallon equal to 4 quarts or 8 pints or 128 ounces than there is to preserve the relation of the yard to the foot.

It does not seem very difficult for anyone who does not care for the square or cubic yard to change the figures to suit himself by the simple process of multiplying by 9 or 27.

Let us therefore perpetuate the yard, along with our other monstrocities in units of measure, as one more monument to Anglo-Saxon obstinacy, politely termed conservatism.—E. W. SCRIPTURE, JR.

Efflorescence on concrete bricks (36-53)

Q—A manufacturer of concrete bricks has had more or less trouble with efflorescence when the bricks are stored in piles in the stock yard after the initial steam curing. He would appreciate advice from other products manufacturers, cement manufacturers or anyone, as to the tendency of different cements to produce efflorescence in products exposed to the weather—are some cements better than

others in this respect, or do all of them have the same potential capacity for causing efflorescence when other conditions are the same.

Compressed air jets as a means of placing concrete (36-54)

Q—Can you give us any data or any helpful comments or suggestions on the use of a compressed air jet in placing concrete? What we feel to be a new idea for the compacting of fresh concrete in forms was discovered quite accidentally during 1935 while replacing some floor slabs and beams in our plant at Akron, Ohio. After a beam section had been poured and spaded, one of the laborers stuck the pipe on the end of an air hose (used for cleaning out the forms) into the freshly poured concrete. The inspector stopped this, feeling it was detrimental to the concrete, but noticed an appreciable shrinkage of the concrete in the forms.

In the pouring of 14 x 24 in. loose bar concrete beams, using 1:2:4 mix with 6½ gal. of water per sack of cement, it was observed that after beams had been thoroughly spaded the application of compressed air caused the concrete to settle (approximately) an additional ¾ in., or a decrease in volume of about 3 per cent.

The only evidence of aeration was at the top surface. No evidence at the bottom or sides of beams after forms were removed. As it is usual for all the impurities, light materials, etc., to come to the top this offers no objection.

Following this, in collaboration with the late Wilbur J. Watson, of Wilbur Watson & Associates, we began to experiment with concrete compacted by compressed air. (Factory air at approximately 60 p.s.i. pressure). The results obtained by use of compressed air alone, hand spading and compressed air, and usual hand spading methods, show a uniform increase in density and compressive strength in favor of the tests using compressed air.

	Hand Rodded	Compressed Air
Average* weight—wet	29.62 lb.	30.64 lb.
Average weight—dry at 28 days	28.50 lb.	30.12 lb.
Average loss in drying	1.12 lb.	0.52 lb.
Average crushing strength p.s.i. at 7 days	1360 lb.	1600 lb.
Average weight of cylinders after curing in moist sand for 28 days	28.80 lb.	30.10 lb.
And same cylinders after being immersed in water for 28 days	29.03 lb.	30.23 lb.

*Averages represent a minimum of ten cylinders in each test.

For the more accurate determination of the properties of air compacted concrete, we ran an extensive series of tests using standard 6 x 12 in. cylinders. In all tests the same number of cylinders were used for each method, and the average shown represents a minimum of ten cylinders for each test.

From the above we find that cylinders compacted by compressed air show the following:

Wet—an increase in weight by	3.5 per cent
Dry—at 28 days an increase in weight by	5.7 per cent
Loss of weight in drying—hand rodded	4.0 per cent
Loss in drying—air method	1.7 per cent
Crushing strength—increased (7 days)	17.6 per cent
Increase in weight due to immersion in water for 28 days—	
Hand method	0.8 per cent
Air method	0.4 per cent

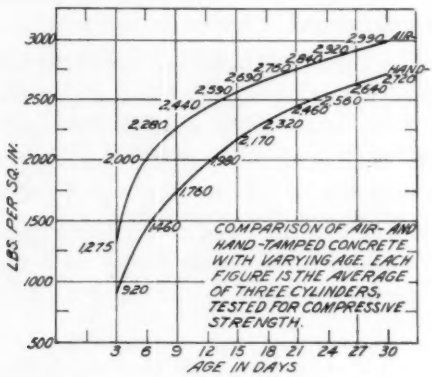
LBS. PER SQ. IN.
AGE IN DAYS
AIR
HAND
COMPARISON OF AIR- AND HAND-TAMPED CONCRETE WITH VARYING AGE. EACH FIGURE IS THE AVERAGE OF THREE CYLINDERS, TESTED FOR COMPRESSIVE STRENGTH.

FIG. 1

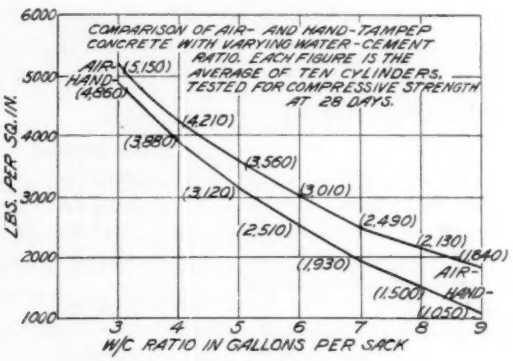
COMPARISON OF AIR- AND HAND-TAMPED CONCRETE WITH VARYING WATER-CEMENT RATIO. EACH FIGURE IS THE AVERAGE OF TEN CYLINDERS, TESTED FOR COMPRESSIVE STRENGTH AT 28 DAYS.
LBS. PER SQ. IN.
W/C RATIO IN GALLONS PER SACK
AIR
HAND

FIG. 2

We have data from various tests used in compiling the above results. Accompanying graphs show: Fig. 1. comparison of air and hand tamped concrete at varying ages, 3 to 30 days, 1-2-4 mix with 6½ gal. of water per sack of cement, and, Fig. 2, comparison of crushing strength at 28 days for air and hand tamped concrete with varying W/C ratios.

We find that by using compressed air we get—greater density; increased crushing strength and less water absorption. These being vital factors in good concrete, it appears that use of compressed air offers a contribution to the art of placing concrete.—M. L. DAVIS.*

Lifting force of freshly placed concrete (36-55)

Q—It is necessary to embed 9 ft. diameter steel pipes in mass concrete. The first lift will be approximately 6 ft. deep from the invert of the pipe, and the pipes rest on concrete piers at 20 ft. centers, approximately 18 ft. high. What lifting force will be exerted on the pipe if a 20 ft. length is poured?†

A—If the concrete around the pipe is *poured* (as the question states), the lifting force will doubtless be very much greater than if the concrete is placed, as mass concrete should be placed at a slump between 1 and 2½ in. and in this case a foot at a time at least 30 minutes apart. The concrete can be thoroughly vibrated with very little added uplift if it is done immediately as the concrete is placed, and care is taken not to penetrate deeper than 6 in. into the preceding layer.

Design practice in such cases is often based on assuming the worst possible condition of full displacement pressure from fluid concrete having a weight of 150 lb. per cu. ft. If answer to the question is made on this basis, the uplift force will be 135,000 lb.

The actual uplift pressure would be influenced by a number of factors. The most significant of these more or less in order of importance within probable limits are: (1) Rate of vertical filling; (2) Consistency of concrete; (3) Concrete and ambient temperature; (4) Setting rate of the cement; (5) Shape and other properties of aggregate; (6) Richness of mix; (7) Placing methods.

A review of ten references describing tests of concrete form pressure revealed that two investigators found 20 to 30 per cent of maximum fluid pressure due to 7½ feet of concrete in the forms, two found 60 to 75 per cent, and six found 100 per cent. This merely indicates that most of them were endeavoring to determine if full fluid pres-

*Engineering Dept., Firestone Tire & Rubber Co., Akron, Ohio.
†From an A. C. I. member in Australia.

sure was possible, and found that it was. The fact that several found *it does not necessarily occur* indicates that a little forethought and engineering control of the placing operations may reasonably be expected to reduce the theoretical maximum pressure by as much as 50 to 75 per cent.—ARTHUR RUETTGERS.*

The near-maximum lifting force to be expected on a large pipe surrounded by fresh concrete would result from considering the concrete as being a self-supporting porous solid saturated with water. If the concrete were to have a slump of about an inch only, as is common for vibrated concrete, full hydrostatic pressure might still be expected on the entire under surface of the pipe, because of the watergain film which usually develops. Hydrostatic pressure should be exceeded only in the local region where the concrete was being tamped or vibrated. A rough calculation shows that the lifting pressure on a 9-ft. diameter pipe, 20 ft. long and 6 ft. deep in concrete, might be about 50,000 lb. It should be understood, however, that such a large pressure would only occur if the concrete were placed quickly and none of it had begun to set. As soon as the cement begins to set, water is drawn in to supply the hydration needs and hydrostatic pressure disappears. It is for the man on the job to decide how much of the under side of the pipe is subjected to concrete which has not begun to stiffen.†

*Bureau of Reclamation, Denver.
†This discussion was submitted by an A. C. I. member who prefers to remain anonymous.

Vol. 36 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
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AMERICAN CONCRETE INSTITUTE

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Curved concrete girders for Golden Gate Bridge approach

Engineering News-Record, Vol. 123, No. 7, Aug. 17, 1939, p. 57 Reviewed by S. J. CHAMBERLIN

The approach viaduct is carried by three curved concrete girders with horizontal radii of 229 ft. and with a maximum span of 78 ft. Spiral reinforcing in the girders was provided to resist the torsional load. Advantages over straight girders were constant spacing between girders, constant overhang, and better appearance.

Electric heat speeds curing of precast concrete joists

H. K. BURCH, *Engineering News-Record*, Vol. 123, No. 9, Aug. 31, 1939, p. 53
Reviewed by S. J. CHAMBERLIN

Lead-sheathed soil cable, wound on sheet metal arbors placed in the hollow metal cores forming the webs of the adjacent joists, formed the heating element. The ten joist mold required nine such heating elements, each 20 ft. long and having 120 ft. of soil cable on 230 volts with a connected load of 7.2 kw. After placement the concrete was heated for two hours and forms stripped in 5 hrs. for a cost of 1.6 kw. per 20 ft. joist. Two pours a day were possible by heating the first and letting the other stand overnight without heat.

Refacing Stevenson Dam

Engineering News-Record, Vol. 123, No. 7, Aug. 17, 1939, pp. 58-60
Reviewed by S. J. CHAMBERLIN

Refacing of the Stevenson Dam (Conn.) featured the construction of a complete set of vertical and herringbone system of drains on the face of the dam behind the new facing to prevent new deterioration. General, but not universal, disintegration of the face of the 20-year old dam was caused by ice formation and erosive effect of water and ice discharge on the concrete of deficient density and durability at many points. The steps in refacing were: (1) chipping the face to hard concrete; (2) cutting the V-shaped drains, covering with elastite and in the case of the vertical construction joints, placing a copper expansion joint over the elastite; (3) placing 4 x 4 in. heavy-gage steel mesh and dowelling; (4) guniting.

Concrete corners in tension

D. B. GUMENSKY, *Engineering News-Record*, Vol. 123, No. 13, Sept. 28, 1939, p. 57
Reviewed by S. J. CHAMBERLIN

Tests were made to determine the relative merit of different types of corner reinforcement of square concrete conduits subjected to hydrostatic pressure. Test specimens were rectangular in cross-section, the jaws of the testing machine pulling on the opposite legs of the angle. Dimensions were made such as to give the same relation between direct tension and bending moment as would exist in a 10-ft. square conduit under hydrostatic load. The first crack appeared in all specimens at about the same tensile load. At ultimate load the corner reinforced by a complete loop and an additional cross-bar with enclosing fillet proved far superior to plain loop and to hook reinforcement.

Testing of concrete cubes of 10 cm. edge of moist concrete

KURT SEIDEL. *Zement*, Vol. 29, No. 38, Sept. 21, 1939, pp. 586-588.
Reviewed by L. T. BROWNMILLER

Frequently it is desirable to substitute a 10 cm. cube for the usual 20 cm. cube in testing concrete of the consistency of damp earth. This article gives the procedure for making the test and compares the results with those obtained on the 20 cm. cubes. The size of the aggregate for the smaller cubes is limited to 30 mm. The molds are completely filled with concrete before compacting by a series of blows with a weight. The method of compacting is specifically outlined. The strength of the 10 cm. cubes must be 15 per cent greater than the strength obtained on the larger cubes.

Contribution to the question of the constitution of portland cement

K. KONANAGI and T. SUDOH. *Zement*, Vol. 28, No. 37, Sept. 14, 1939, pp. 563-568.
Reviewed by L. T. BROWNMILLER

This paper contains a report on the constitution of tricalcium silicate in Portland cement, as revealed by x-ray and microscopic studies. Laboratory preparations as well as Portland cement clinker were used by the authors for their examinations. From the studies the authors found that tricalcium silicate forms limited solid solution with magnesia and tricalcium aluminate. The x-ray patterns of such solutions are identical with the patterns of alite in Portland cement. About 1.0-1.5 per cent of magnesia is the limit of solution in tricalcium silicate.

Lean and wet slag mix makes good concrete

JOHN K. HARRIS, *Engineering News-Record*, Vol. 123, No. 9, Aug. 31, 1939, pp. 47-48
Reviewed by S. J. CHAMBERLIN

Slag concrete in the foundations of the Republic Steel Corp. mill at Cleveland, placed two years ago, is seemingly of good quality despite a lean and wet mix. The coarse aggregate was well graded, blast-furnace, air-cooled slag requiring a comparatively high percentage of fines. The materials used per cu. yd. of concrete were: 20 cu. ft. of coarse slag (1½ in. and 1 in. maximum); 4 cu. ft. of fine slag (⅜ in. to dust); 14 cu. ft. of natural sand; and 4.6 sacks of cement. Excepting for floor slabs no curing was done. Concrete piles, formed by placing the concrete in the shells immediately after the withdrawal of the driving mandrel, attained proper set despite adjacent continuous vibration and ground quaking. An unreinforced submat 4 in. thick placed just below top of piles proved a construction expedient in holding the piles laterally and providing a deck for placing the 20 in. thick reinforced mat.

Making large aqueduct pipe

Engineering News-Record, Vol. 123, No. 9, Aug. 31, 1939, pp. 66-69 Reviewed by S. J. CHAMBERLIN

Heavy concrete pipes in sizes up to 12½ ft. diameter are being produced for the new pressure aqueduct of the Boston Metropolitan District water supply system. The steel bell-and-spigot joint rings are rolled, welded at the closing seam, and then stretched in an hydraulic press to size and form to fit within a tolerance of $\frac{1}{32}$ in. on the diameter. A complete pipe shell is made from sheet steel automatically welded and rolled into pipe form. The joint rings are welded on and one elliptical and one circular reinforcing spiral then wound on the shell. The inner and outer concrete forms and the steel assembly are placed vertically on a circular base ring, a top ring acting as a casting platform. The concrete is poured with the aid of a rotary hopper and distributor chute. Vibrators are clamped to the outside form. Curing is accomplished by moist steam fed into the inner form until the forms are removed the next morning, further steam curing under canvas hoods for two days is followed by water-spraying for twelve days. The rich concrete mix of low water content attains strengths of 5,000 p.s.i. at 28 days. The pipe, protected with radial struts, is delivered by low-body trailer trucks, and placed by railway derrick car on a track in the trench or a gantry derrick spanning the trench.

Tests of reinforced concrete slabs subjected to concentrated loads

F. E. RICHART and R. W. KLUG. *Bulletin 314*, Eng. Exp. Station, University of Illinois, June 1939. AUTHOR'S REVIEW

The bulletin describes tests of two reinforced concrete slabs, 6 ft. 8 in. in span and 20 ft. wide, and 18 smaller slabs, 5 ft. square, under concentrated loading simulating highway wheel loads. In all tests, extensive strain measurements were taken at loads which did not crack the concrete. These were followed by similar observations at higher loads, and finally the slabs were loaded to failure. Careful comparisons were made between measured strains and deflections and computed values. In the tests of square slabs, a study was made of the effect of size and shape of bearing area for load application.

Failure of the slabs generally occurred by punching out of a truncated cone of concrete under the load. A comparison of measured and computed strains in the reinforcement showed fair results. In the large slabs, the highest measured strains were about equal to the computed values, except directly under the load where the measured values were about four-fifths of the calculated ones. In the square slabs the agreement was closer. The tests indicate a fairly good check on steel stresses between analysis and tests.

Tests of plaster-model slabs subjected to concentrated loads

N. M. NEWMARK and H. A. LEPPER, JR., Bulletin 313, Eng. Exp. Station, University of Illinois, June, 1939 AUTHOR'S REVIEW

Model slabs of pottery plaster were tested under various conditions of loading when supported in different ways. By means of the tests approximate values of the maximum tensile stress for elastic behavior of the material were obtained. Comparison was made with stresses computed by the mathematical theory of elasticity and a fairly good agreement was found between the theory and the test results.

In previous tests of plaster, difficulty was encountered due to the presence of numerous tiny air bubbles in the material. It was found that high frequency me-

chanical vibration applied during mixing of the plaster caused practically all of the entrapped air to escape and produced a dense material with no visible cavities or voids.

Although the behavior of a reinforced concrete slab at ultimate loads is different from the behavior of a plaster model slab at failure, there is considerable similarity in behavior between the plaster slab up to rupture and the reinforced concrete slab at working loads. Thus the loads causing failure and the manner of failure of the plaster slabs should give qualitative indication, at least, of the action of reinforced concrete slabs.

Moments in simple span bridge slabs with stiffened edges

VERNON P. JENSEN, University of Illinois Engineering Experiment Station, Bulletin 315, Aug., 1939.
AUTHOR'S ABSTRACT

The bulletin gives curves and tables of moments, determined analytically, for the bridge floor slab which spans in the direction of traffic, is simply supported on two opposite edges, and is stiffened by curbs or beams on the other edges. The span is limited to a maximum of 30 ft. The loadings considered are the following: a concentrated wheel load at any point on the transverse center line of the slab; combinations of 4 wheel loads on the transverse center line caused by passing trucks; a uniformly-distributed load on the slab, the weight of slab and pavement; a uniform line load on the edge members, for example, the weight of curb and handrail. For each of these loadings the moments determined are the following: the moments M_x and M_y under an inside wheel which is placed for convenience at the center of the slab; the moment M_x under an outside wheel near the edge of the slab; the moment at the center of the edge member. The effects of the ratio of the sides of the slab and of the relative stiffness of edge members are shown.

Illustrative problems are solved. The required area of transverse steel in the slab is found to be somewhat higher than has been previously indicated. This is accompanied by an increased steel requirement in the curb and a slight reduction in longitudinal moment in the slab. As a result a slight reduction in the thickness of slab is indicated for the larger spans and ordinary widths.

Submerged dummy joints

JOHN J. CASEY, *Pacific Road Builder and Engineering Review*, July, 1939, pp. 27-28.
HIGHWAY RESEARCH ABSTRACT

As a means of eliminating some of the undesirable features of dummy contraction joints, the city and county of San Francisco has experimented with a submerged dummy joint. This joint consists of one-half in. by four in. clear pine board placed in two 12 ft. lengths across a 24 ft. pavement which is 8 in. thick. This board rests on the subgrade and is held in place by nailing to seven 20 in. pine stakes spaced on alternate sides of the board at 4 ft. intervals. Joints are spaced at 20 ft. intervals. The concrete was placed, spread and finished by machine with no special attention given to the dummy joints other than the average care used to prevent displacement of the pre-placed wooden strips.

The resulting concrete surface is smooth and has very good riding qualities. In 24 hrs. the location of these dummy joints showed up as a slightly irregular hair-crack across the pavement. No intermediate contraction cracks developed.

This experimental section was on a new street with two 24 ft. roadways divided by 5 ft. strip. One side was built with the surface type of dummy joint, and the other with the submerged type. Both sides were then covered with a two-inch topeka wearing surface. It is reported that there is a definite improvement observed

in the submerged joint section in the first months of use. No asphalt joint filler was present to bleed into the topeka wearing course and no unevenness was present in the concrete base at joint locations to influence the surface finish.

Crackless concrete for Hiwassee Dam

Engineering News-Record, Vol. 123, No. 11, Sept. 14, 1939, pp. 69-72 Reviewed by S. J. CHAMBERLIN

In constructing the high-gravity Hiwassee Dam the T. V. A. has taken especial precautions in the elimination or reduction of cracks in the concrete. Preventative measures are: (1) use of low heat cement (maximum percentages, C_2S-35, C_2S-65, C_3A-7, C_4AF-20) with a required fineness between 1700 and 2200 sq. cm. per gr., and minimum strengths of 500 p.s.i.—3 days, 1000 p.s.i.—7 days, and 2000 p.s.i.—28 days; (2) use of low cement content (0.8 bbl. per cu. yd. for mass concrete); (3) use of low lifts of pours (2½ to 5 ft.); (4) long intervals between pours (1 day per ft.); (5) refrigeration of mixing water in summer (35-45 degrees F.); (6) heating of mixing water to 50 degrees F. in winter; (7) washing of aggregates; (8) use of diagonal keys in vertical contraction joints in the lower third of dam to provide shear resistance; (9) careful clean-up of construction joints (two applications of air and water jet); (10) curing of the concrete for three weeks by water spray. Ragged edges, at horizontal construction joints and vertical contraction joints, are eliminated by the use of a 1¼ x 3¾ in. chamfer excepting at the spillway section. The gradation of aggregate is a straight line on log paper from 6 in. maximum size down to minus 4-mesh, 3 per cent passing a 100-mesh. The mix for mass concrete by weight is $1:2.79:9.34,\frac{w}{c} = 0.8$. From the three mixers the concrete goes by train to the 7½ cu. yd. cable way bucket. Measuring instruments, strain meters and stress meters, are being installed in the dam to study service behavior. Samples of the material around the instruments are being sent to a research laboratory for further study.

Air raid precautions

Indian Concrete Journal, Vol. 13, No. 9, Sept. 1939. Reviewed by J. C. PEARSON

The September issue of this journal is devoted almost entirely to a discussion of air raid precautions, all phases of the subject being covered from sizes, kinds, flights and effects of bombs dropped from the skies, to the design, location, construction and equipment of all kinds of shelters. Aside from deep subterranean structures which lend themselves naturally to maximum protection, the construction of wholly bomb-proof shelters is out of the question for most of the inhabitants of an exposed area, if only for the reason that the cost of such shelters is in most cases prohibitive. The general principle involved for protection of the multitude is that least injury will be caused if small shelters are most widely scattered. These shelters are not designed to be proof against direct hit, but are intended to afford sufficient protection from splinters, concussion, debris, and gas, when heavy bombs explode not nearer than about 50 ft. away. These shelters are also proof against incendiary bombs of small size. Naturally, reinforced concrete is the preferred material for such shelters, whether built above or below ground, in basements or beneath or along side of existing buildings, or apart from buildings in parks and gardens. Many designs and types are illustrated; one of the simplest so far as ready purchase and quick installation are concerned is one or more lengths of precast reinforced pipe 72 inch or 90 inch in diameter. These are laid horizontally, with square end closures, are fitted with

simple seats of the "settee" type along either side of a central walkway, and may have entrances or exits at both ends with provision for gas-tight doors. Many private installations are, of course, designed to serve a useful purpose in times of peace, such as storage for vegetables, tools, valuables, etc.

A new method of proportioning of concrete aggregates

F. MOGENSEN, *Cement och Betong*, July, 1939, pp. 3-11 Reviewed by BENGT FRIBERG

The paper describes a noteworthy departure from the customary mechanical sieve analysis gradation curves for proportioning of aggregates, with increased clarity in regard to desired and obtainable size distribution. The proposed method establishes for each aggregate the relative mass distribution, expressed as the part of the mass retained within each relative interval in grain size, plotted against the particle size in logarithmic scale as abscissa. The relative mass distribution M for V per cent of weight retained within grain size k_1 and k_2 and mean size k_m is expressed as

$$M = 100 \cdot V \div \frac{k_2 - k_1}{k_m}$$

a continuous curve is drawn through the stepped values obtained for the ordinary screen series. As in the diagram equal surfaces represent equal amounts of aggregate, the relative mass distribution curve gives a clear illustration of variation in and characteristics of a given aggregate.

The mathematical ideal mass distribution curve for minimum amount of voids follows the curve

$$M_k = c \cdot k^{0.6}$$

to $M = 60$ for the maximum particle size and has the same shape within wide limits of particle sizes so that the ideal curve can be drawn as soon as the maximum particle size is known. Proportioning is done so that the M-curves for each of the participating aggregates and the ideal curve are drawn. When the M-values are reduced in the same proportion as the trial aggregate proportioning and the sum-curve (considering overlapping aggregate sizes) is drawn, it should as nearly as possible coincide with the ideal curve for the maximum size aggregate used.

Comparison of trial proportions and between different aggregates is clear and the correct proportioning is quickly established. Typical diagrams and examples of use are illustrated in the paper.

Future development of highway equipment

W. VANCE BAISE, State Highway Engineer, North Carolina. (Excerpt from a paper delivered at the 25th Convention, A.A.S.H.O., Oct. 12, 1939.) HIGHWAY RESEARCH ABSTRACT

Amazing development has been accomplished in highway equipment during the past 25 years through the cooperative efforts of contractors, engineers and manufacturers. The factors exercising the greatest influence on equipment improvements have been the competition among contractors to reduce costs by imcreased capacity and speed of operation, competition among manufacturers to meet the demand for this better equipment, and the determination of highway engineers to accomplish more work per highway dollar.

Price changes over a period of years in North Carolina for a few principal items show the results which have been attained in spite of the fact that the cost of common labor has increased, and in spite of higher engineering specifications. The Public Roads Administration has found that such reductions are general over the U. S.

ITEM		1920-23	1939
Common excavation	Cu. Yd.	$ 0.50	$0.20
Borrow excavation	Cu. Yd.	0.40	0.20
Concrete pavement	Cu. Yd.	14.00	8.75
18 in. reinforced concrete pipe	Lin. Ft.	2.12	1.53
Reinforcing steel	Lb.	0.0588	0.0482

As to future developments, it may be prophesied that:

(1) There is a general trend to heavier and more powerful equipment. This is especially true with such units as the motor grader and pull-type grader, travel mix plants, etc. However, it is not true of such items as pavers, power shovels and scrapers.

(2) It is a foregone conclusion that diesel power will play a prominent part in equipment development.

(3) The adoption of pneumatic tires for transporting is predicted, and as a consequence the use of crawler tracks will be limited to lugging (either pulling or pushing), and to use in extremely unstable soil conditions. Quantities of material have been transported on crawler tracks, and in many cases those loads required the traction of a crawler track to get them started, but once the load is started a faster and cheaper transportation can be obtained by having both the load and the motive power on pneumatic tires.

(4) The adoption of power controls has been general and will probably be extended to eliminate additional manually performed operations.

(5) More flexible power transmission through the hydraulic turbine drive will probably find its place in highway equipment. It has already been introduced in cars and in industrial power.

(6) Equipment for the various types of stabilization will be improved. There are still many cumbersome operations in connection with stabilization and road mix projects. The success of road mix jobs is still too dependent on weather conditions.

(7) The adoption of special alloy metals to reduce weight and to increase strength may be expected.

The antique art of mortar making

F. Mueller-Skjold, *Tonindustrie-Zeitung*, Vol. 63, No. 73, p. 824 (Sept. 11, 1939)
Reviewed by A. U. Theuer

The author describes the results of several studies, including his own, on the nature and properties of Roman concretes. A criticism of much written in the past is voiced on the grounds that overgeneralizations and conclusions were drawn from single samples. A study by Gruen and Biehl, using samples taken from the Cologne aqueduct, is discussed first. This structure, with walls averaging 12 in. thick, was built in several courses of concrete. The cementitious material was lime originating in the Soetenich deposits. The first course consisted of a fine sand-lime mortar. Crushed limestone was added to this mixture for the second course. The aggregate distribution curve is said to follow closely the recommendations for current (German) practice. The compressive strength of cubes cut out of the structure are said to average 1620 p.s.i. Just as in samples taken from the Roman Forum and Hadrian's monument, studied by these investigators, the proportion of lime to aggregate is from 1:2.6 to 1:2.7 parts by volume. For the two Roman structures cited, however, the aggregate consisted of quartz and brick dust.

The author's studies are confined to wall finishes and samples are all of Italian origin. The Casa del Lirarei at Pompei which was refinished and redecorated just prior to the eruption of Vesuvius 79 A.D. provides the principal source of material for the study.

Coursed structure again characterizes this work and the individual courses are said to be easily traceable by eye, and just as described by Vitruvius. Polished thin-sections were used for detailed study. The first coat is dark grey-brown in color and was found to correspond roughly with the mortar used in the Forum as described by Biehl. Averages for the analysis made of seven samples gave the following proportions: 20 to 25 per cent brick dust, 15 to 20 per cent quartz sand, 55 to 65 per cent volcanic stone particles. The lime is described as a high strength white lime, 97 per cent calcium oxide, well-burned. Lime pockets were absent in the samples taken for this work and in this respect differed from those taken by Biehl from the Forum. Aggregate distributions indicated 50 per cent passing the 1.5 mm sieve and retained on the 0.5 mm sieve. The finish coat is light in color and the aggregate is a crushed *marble*, i. e., cyrstallized calcium carbonate. After set the coating contains from 96 to 97 per cent of calcium carbonate, leading to difficulties in further analysis. As mineralogical-optical method indicated, the proportions of marble derived from aggregate as against that derived from the lime paste to be in the ratio of one to one. An intermediate coat was used to smooth off depressions left in the undercoating. This coating seldom averaged more than 2 mm and was similar to the finish coat except for the aggregate distribution. The aggregate of the finish coat was from 1.2 to 2.0 mm and over. In contradiction to modern practice where an increasingly fine aggregate follows from the wall out to the surface the Romans follow the rule: medium, coarse, fine. It is only the final finish coat that contains very fine aggregate and this coat is characterized by great strength.

The author then discusses the application of the results of the studies in current practice. Considered as entirely impractical is the process of soaking the lime for two or three years prior to use. The principles, however, of using finishes made up of varied courses and of using crushed limestone for aggregate are both matters worth considering. Another essential point, i. e., the use of brick dust, has led to considerable disagreement. Some investigators ascribe pozzolanic activity to this material. The author, from his studies, believes some faint indication exists for this belief. He advances another reason for its general use, however, and that is the possibility that the particles, especially the coarser ones, act as water reservoirs during setting.

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Proposed Recommended Practice for Measuring, Mixing, and Placing Concrete*

Reported by Committee 614

LEWIS H. TUTHILL, CHAIRMAN

INTRODUCTION

This report has been prepared in response to an expressed need for an outline of good practices for measuring and mixing the ingredients for concrete and for placing the finished product.

At the outset the Committee faced the question whether it should describe "common practices" or "best practices." Since this report is a recommendation, and not a specification, it is believed that the interests of progress in the development of concrete construction will be better served if the best methods known are outlined.

On small jobs, or in the more competitive uses of concrete such as building construction, practices somewhat more relaxed than those here recommended may be economically justified. The profitable degree of such deviation must be determined by those in charge of each job, in view of the conditions which obtain. In general, the Committee has no evidence that good concrete operations cannot be performed as economically as poor ones or that the methods here recommended, where applicable, will result in higher ultimate costs than other methods. The refinements recommended are intended primarily to improve uniformity and to eliminate segregation in

*This report (in its present form) was received by the Institute Jan. 8, 1940. In all major essentials it has been approved by 11 members of Committee 614, two members not voting and three members asking further revision. This report is released by the Standards Committee for publication as information and for discussion and on that basis will go on the program of the 36th Annual Convention Feb. 27-29. Members of the committee in addition to Chairman Tuthill are John G. Ahlers, C. F. Ball, E. W. Bauman, Sam Comess, R. B. Crepps, Lion Gardiner, Byron A. Hill, C. S. Johnson, A. A. Levison, O. G. Patch, Donald D. Reynolds, R. W. Spencer, H. F. Thomson, I. L. Tyler, R. B. Young.

aggregates and concrete. Where properly executed, the methods described have resulted in smoother operation and any additional costs resulting from them have been offset by consequent higher production rates.

It has been assumed that whoever will give serious consideration to these recommendations will have a reasonable knowledge of the ordinary practices and preparations required in concrete work. For that reason, many routine instructions for measuring, mixing, and placing concrete are omitted. Since the specific objective of these recommendations is maximum uniformity, homogenity and quality of concrete in place, special consideration is given practices designed to accomplish that end. In order to portray more clearly certain of the principles involved and their application to concreting operations, a drawing is reproduced as a part of these recommendations, illustrative of some examples of good and bad practice.

For further discussions of the standard aspects of measurement, mixing, and placing, the reader is referred to the 1937 Progress report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete and to the particularly excellent and specific document of recommended practice adopted by the American Association of State Highway Officials relating to Measuring, Mixing, and Placing of Concrete for Use in Concrete Pavements. With the recommendations of those reports, A. C. I. Committee 614 is, in general in accord, although a few exceptions in detail or in emphasis may be noted.

MEASUREMENT

Essential requirement

Equipment should be used which is capable of performing accurate measurement. Materials should be handled, and measuring operations performed in such a manner that satisfactory reproducibility of the selected batch assembly and aggregate grading is obtained batch after batch. The objective is ultimate uniformity and homogenity of the concrete.

Aggregates

The number of separate sizes of coarse aggregate which should be used, to reduce segregation in handling to a practical minimum and to permit reasonably close control of the grading from batch to batch, is dependent upon the maximum size of the material. For particles larger than 1 inch, the ratio of the maximum to the minimum designated sizes should not exceed 2; for coarse aggregate particles finer than 1 inch, this ratio should not exceed 3. The permissible combined

tolerance of undersize and oversize material in each designated size of aggregate should not exceed 10 per cent, and handling operations should be such that variation in the portions of such material is held to a practical minimum.

The maximum size of sand should not exceed that which will pass a screen having ⅛-in. openings if it is screened and handled dry. The standard No. 4 sieve (3/16-in. openings) represents a suitable maximum size for wet screened sand which is to remain damp throughout all handling and batching operations, except when the use of the smaller size is desirable to control the percentage of pea gravel (about No. 8 to ⅜-in.) in the mix. When two sizes of sand are used to obtain proper grading, no attempt should be made to blend them during stockpiling or loading of cars or trucks. With adequate equipment and ample supervision and inspection, good results are obtainable, when the fine aggregate is produced, by blending the different sizes of sand as they flow into a common stream from regulating gates or feeders. Such methods can, however, produce very irregular results. For the majority of work, separate batching of the fine and coarse sand is preferable since it will produce consistently uniform results under the wide range of common plant and job conditions.

The production or purchase of sand should be controlled insofar as possible to minimize variations in grading, particularly in the finer sizes. In plants where sand is washed, care should be exercised to avoid removal of fines essential to good grading and to plastic, workable concrete. For sand passing a ⅛-in. screen, 20 per cent finer than 50 mesh is considered close to an ideal percentage of fines for average concrete work. Lean mixes are benefited by ample percentages of such fines while rich mixes are improved with a smaller percentage of these sizes.

Varying amounts of water in aggregates commonly contribute to lack of uniformity. Effort should be made to maintain a uniform moisture content in the aggregate supplied for batching. For this reason aggregates from dry deposits, if not high in absorption, may preferably be prepared and handled dry when washing is not essential. Before placing the material in batcher bins, wet aggregates should be drained until, by actual test, there is little difference in the moisture content of the wettest and driest portions of the materials. Adequate drainage will require at least 8 hours and additional time may be necessary, depending on the grading of the material and the efficiency of the drainage system provided. On large jobs when thorough drainage is difficult, an electric moisture indicator in the sand batcher and a consistency meter on the mixer will permit rather close control

of consistency, even when there are wide fluctuations in aggregate moisture.*

Aggregates should be handled in stockpiles and bins in a manner producing minimum segregation of sizes. Stockpiles should be permitted only when built up in layers no thicker than those resulting from truck loads dumped on the same plane. When stockpiling sand and pea gravel, segregation may be further avoided if moisture contents of at least 5 and 2 per cent, respectively, are maintained. Coarse aggregate should be removed from stockpiles, by clamshell bucket or other means, in horizontal layers of such width that aggregate is not permitted to run down the slopes at the edge of the pile. Storage bins should have the smallest practicable equal horizontal dimensions. Round bins are preferable. Bins should have one outlet only located at the center of a bottom sloping to it from all directions at not less than 50 degrees from the horizontal. They should be charged by material falling vertically directly over the outlet. They should be kept as full as possible at all times to avoid breakage and grading changes as the bins are emptied.

Reference is made to the sketches, a part of these recommendations, for illustrations of preferred practice in the handling of aggregates. Unless precautions are exercised in the selection and handling of aggregates, refinements in measuring, mixing and placing are nullified.

Batching Aggregates

For jobs of from 2,000 to 5,000 cu. yd., the cost of hand batching soon justifies a simple weigh batcher installation, into whose bins aggregates may be delivered directly. As the size of the job increases, the saving in materials and the prevention of contamination by dirt or by other sizes adds considerably to the balance in favor of bins as opposed to dumping materials on the ground. As the size of the work increases still greater refinement should be required.

From 2,500 to 10,000 cu. yd. jobs, cumulative manual weigh batching and from *10,000 to 25,000 cu. yd.*, cumulative automatic weigh batching are recommended. For jobs of over *25,000 cu. yd.*, requirement of individual automatic weighing of the aggregate is justified by the improved perfection of uniformity of measurement. (See preferred sketch.) If good inspection and dependable plant men are available, it may not be economically justifiable to require new equipment for the smaller jobs in the foregoing brackets.

Individual automatic and cumulative manual aggregate batchers should be charged through easy operating clamshell or undercut

*Engineering News-Record, Page 52, July 11, 1935.

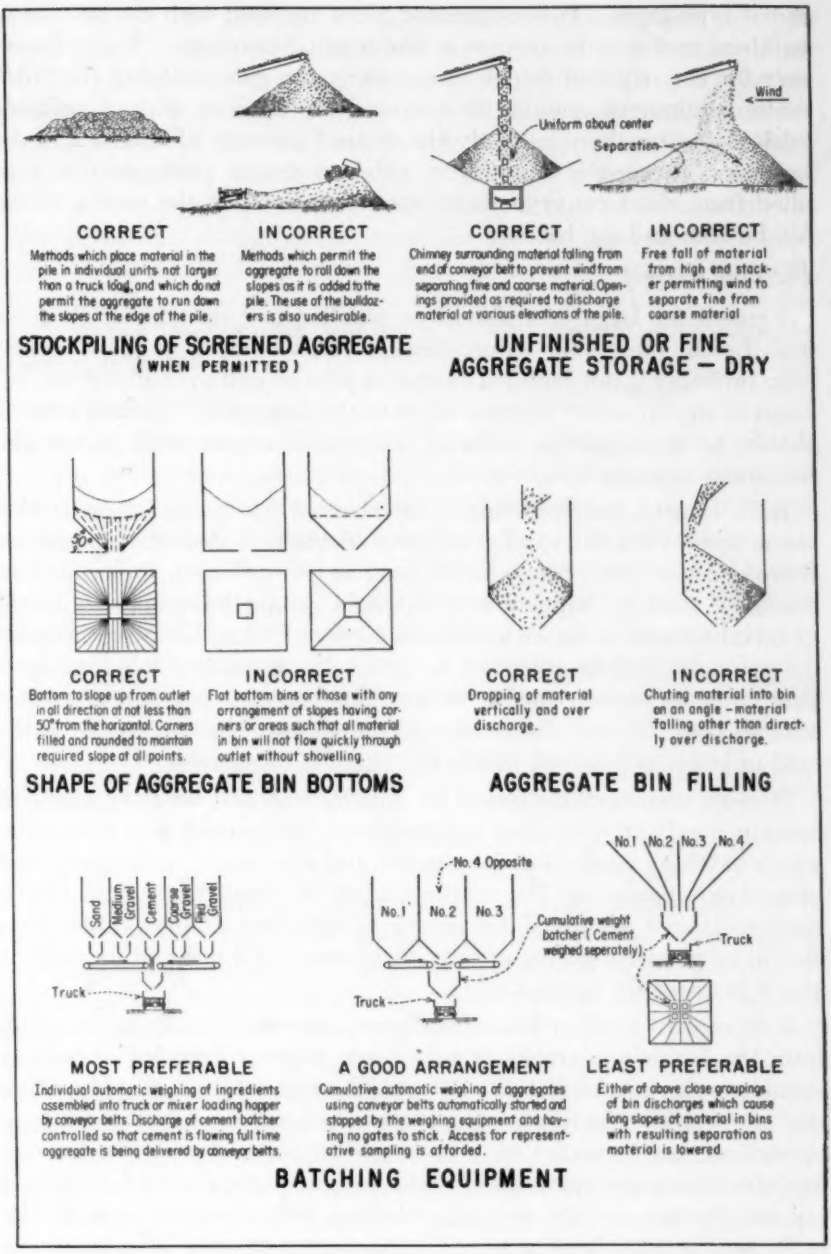
Wind
Uniform about center
Separation
CORRECT
Methods which place material in the pile in individual units not larger than a truck load, and which do not permit the aggregate to run down the slopes at the edge of the pile.
INCORRECT
Methods which permit the aggregate to roll down the slopes as it is added to the pile. The use of the bulldozers is also undesirable.
CORRECT
Chimney surrounding material falling from end of conveyor belt to prevent wind from separating fine and coarse material. Openings provided as required to discharge material at various elevations of the pile.
INCORRECT
Free fall of material from high end stacker permitting wind to separate fine from coarse material
STOCKPILING OF SCREENED AGGREGATE
(WHEN PERMITTED)
UNFINISHED OR FINE AGGREGATE STORAGE – DRY
CORRECT
Bottom to slope up from outlet in all direction at not less than 50° from the horizontal. Corners filled and rounded to maintain required slope at all points.
INCORRECT
Flat bottom bins or those with any arrangement of slopes having corners or areas such that all material in bin will not flow quickly through outlet without shovelling.
SHAPE OF AGGREGATE BIN BOTTOMS
CORRECT
Dropping of material vertically and over discharge.
INCORRECT
Chuting material into bin on an angle – material falling other than directly over discharge.
AGGREGATE BIN FILLING
Sand
Medium Gravel
Cement
Coarse Gravel
Fine Gravel
Truck
No. 4 Opposite
No. 1
No. 2
No. 3
Truck
Cumulative weight batcher (Cement weighed seperately)
No. 1
No. 2
No. 3
No. 4
Truck
MOST PREFERABLE
Individual automatic weighing of ingredients assembled into truck or mixer loading hopper by conveyor belts. Discharge of cement batcher controlled so that cement is flowing full time aggregate is being delivered by conveyor belts.
A GOOD ARRANGEMENT
Cumulative automatic weighing of aggregates using conveyor belts automatically started and stopped by the weighing equipment and having no gates to stick. Access for representative sampling is afforded.
LEAST PREFERABLE
Either of above close groupings of bin discharges which cause long slopes of material in bins with resulting separation as material is lowered.
BATCHING EQUIPMENT

FIG. 1—CORRECT AND INCORRECT HANDLING OF AGGREGATES IN STORING AND BATCHING

radial type gates. Power-operated gates are used with the automatic batchers and may be used with the manual batchers. Where necessary for the required degree of accuracy, the gate charging the automatic equipment should be arranged to operate with a suitable "dribble" after there is nearly the desired quantity of material in the batcher. Automatic cumulative batchers should preferably be supplied from short conveyor belts operating between the center of the bin bottom and the batcher.

Batching Cement

Cement for large jobs should be handled in bulk and weighed for each batch, preferably automatically. New equipment for this purpose probably is not justified except on jobs of well over 10,000 cu. yd. Cement should never be weighed with the aggregate. Sacked cement should be measured in units of not less than one sack unless the fractional bags are weighed.

Bulk cement batchers should be charged by means of controlled screw conveyors or equally effective devices. Moisture should be thoroughly removed from the air used in jets to loosen the cement in storage. Cement hoppers and batchers should have smooth metal or metal covered sides with rounded corners. Silos, bins, and hoppers for cement should be governed by principles outlined above for aggregate bins. Control should be available on the batcher discharge gate so that, in event of over-run, the excess cement may be withheld unless it is removed before the batch is discharged.

Modern specifications issued by government and highway agencies contain excellently detailed specifications for manual and automatic weigh batching equipment for cement and aggregate. Ordinarily, the present equipment on the market, when in good mechanical condition, will have approved tolerances in weighing ability. Operation should be required within a degree of accuracy of 1 per cent for cement and 2 per cent for aggregate.

Bulk cement is often lost or scattered indescriminately in dropping from the batcher to trucks or batch cars below. Free fall of cement should never be permitted. A typical good method of preventing this is to enclose the discharging cement in a narrow canvas boot long enough so that its outlet may be buried in the cement when loading portable batch compartments. Such compartments should contain a separate section for the cement, attached to and operating with the individual batch release gate. The boot may be used as a tremie or the cement may be eased into the compartment by controlling the amount of kinking or doubling back of the canvas at the lower end

of the boot. The canvas may then be lifted, shaken, and dropped into the next compartment. A telescopic rubber hose drop-chute from the cement batcher is also available for this purpose. If a separate section is not provided for the cement in each batch compartment, or the cement is not enclosed in the aggregate, tarpaulin covers should be required during transportation. For placing bulk cement into a batch hopper for a stationary mixer, a pipe of suitable size to hold the cement should extend from the batcher discharge to a level near the bottom of the hopper. After covering the end of this with the aggregate, the cement may be discharged. It will then enter the mixer without loss or dust, and it will be well distributed through the entering aggregate.

Water Measurement

Water measurement on the larger jobs justifies the recommendation that for pavers and other portable mixers, there be used only automatic meters of proved mechanical dependability, or automatic measuring tanks of the vertical cylinder center-siphon discharge type, capable of routine measurement within an accuracy of 1 per cent under all operating conditions. For central mixing or batching plants, either of these devices or automatic weigh batchers of proved dependability and accuracy within 1 per cent should be required. Tanks should be permitted only as an auxiliary part of the automatic meter or weigh batcher, but not as a means of measurement, except in the form of a vertical cylinder with center-siphon discharge.

The Small Jobs

For the small job, usually less than two to five thousand cubic yards, concreting operations are likely to be so irregular that many of the foregoing preludes to uniformity in the concrete, as far as aggregate preparation and handling are concerned, are often next to impossible of attainment. It is well to keep them in mind, however, as standards to be used wherever possible. Cement should be handled and measured in sacks. Mixers should be of sufficient capacity and batches of such a size that no fractional bags of cement are required unless they are weighed for each batch. Water measuring equipment of expensive refinement may not be economical for the small job, but the use of simple equipment that is accurate, positive and dependable is justified and should be required. A manually operated standard disc water meter, of capacity according to the size of the mixer, equipped with a vertical-face, set-back register is not expensive and is recommended where water temperatures do not exceed 100°F. A vertical cylinder tank with center-siphon discharge is equally satis-

factory, and should be used if water temperatures are greater than 100°F. Aggregates should be weighed. In a metropolitan area where weigh batched aggregates or plant controlled ready mixed concrete is available, one of these should be used, even at a reasonable premium, in preference to handling and measuring the materials on the job. Where such services are not available, the quantities for each batch should be weighed in wheelbarrows or buggies on platform scales now generally available for this purpose on small jobs.

General

If standards of uniformity, such as are readily obtainable with modern batching equipment, are to be maintained uniformly throughout an operation, there is greater need for emphasis of correct procedure in incidental operations than on the actual weighing of ingredients. Care should be exercised that batches accurately assembled arrive as uniform batches in the mixer. These sources of error should be avoided: (1) Over-lap of batches in loading and discharging multiple batch trucks and cars. (2) Loss of material in transferring batches to the skips of portable mixers. (3) Loss or "hang up" of a portion of one batch, by including it with another, when dry batches are transferred by means of belts and hoppers. All such damage to the correctness of the quantities in each assembled batch should be strictly avoided and methods should be approved or discarded accordingly.

MIXING

Essential requirements

Thorough mixing is, of course, a first essential. Equipment and methods used shall be such as to insure uniformity of strength, and uniformity of consistency, cement and water content, and aggregate grading from beginning to end of each batch *as discharged.* For concrete work of the highest general quality, mixing equipment should be used which is capable of handling concrete suited to the particular work, from the standpoint of its placeability and workability *in the forms* by means of vibration. Mixing equipment requiring, for efficient operation, mixes containing larger proportions of sand, cement, or water than indicated for best results on the basis mentioned is not recommended.

The Mixer

The mixer should consist of a revolving drum having such a combination of blade arrangement and drum shape as to insure an end to end exchange of the materials parallel to the axis of rotation as well as a rolling, folding, or spreading movement of the materials over on itself as the drum is rotated. The rotating drum should be operated

at the speed designated by the manufacturer of the mixer, unless a reasonable change of speed definitely shows better results.

Any mixer leaking mortar or causing waste of materials due to faulty size, shape, or operation of charging equipment should be taken out of service immediately until satisfactory repairs and improvements have been made to eliminate such waste. Hardened concrete or mortar should not be permitted to accumulate on the inner surfaces of mixing equipment. Worn blades should not be continued in service when they materially decrease mixing efficiency.

Size of batch

The size of the batch should not exceed the manufacturer's guaranteed capacity of 10 per cent more than the rated capacity of the mixer. The rated capacity should be exceeded only when: (1) It results in no loss of mortar during mixing, and (2) tests show the larger batch to be properly mixed. When, to shorten time of discharge or to reduce segregation due to retention of coarse aggregate, it is desired to retain a portion of the batch in excess of the normal "hold back," the amount of material held back should be taken into account in fixing the size of the batch in relation to the capacity of the mixer.

The charging operation

All ingredients of the mix should enter the mixer promptly and as uniformly as possible at the same time. It is particularly important in charging stationary mixers that the solid materials be arranged in the charging hopper in such a manner that no one of them enters separately but that proportional amounts of each (particularly of cement and sand) will be in all parts of the stream of material as it flows into the mixer. To avoid "gumming" and for greater uniformity, cement should never be charged separately, especially ahead of the aggregates but should be enclosed within the other materials. Water for the batch should be released first and continue to flow while the solids are entering the mixer, and should have completed flowing shortly after the last of the batch has entered the drum. This flow should not continue for more than the first 25 per cent of the mixing time, and should be stopped as much short of this as the apparent requirements of good operation of the particular mixer will permit. Where meters are used, and particularly if water pressure is low, auxiliary tanks, into which the quantity of water for the batch may be measured in advance, equipped with a discharge pipe to the mixer of such a size as to insure the foregoing rate of delivery, may be necessary.

Mixing time

The net mixing time for a one-yard or smaller mixer when mixing for gravel concrete of medium consistency containing 1.5 bbl. or more of cement per cu. yd. should not be less than 1 minute under the best conditions. For concretes containing less cement, mixed to drier consistencies, or containing an unusually harsh mixing aggregate, time should be increased 25 per cent to 50 per cent according to the degree of departure from concretes of the most readily mixed type.

For mixers larger than one cu. yd. capacity, particularly when mixing mass concrete, a nominal minimum net mixing time of 1 minute plus ¼ minute for each additional cubic yard of capacity should be required. The use of mixers unable to pass subsequently prescribed tests for mixing efficiency after twice this period of mixing and when charged indescriminatingly, should be prohibited. The total net mixing time for mixers meeting this test should be the foregoing minimum (determined by the size of the mixer), plus such additional time as necessary to bring the batch within the requirements of the mixer efficiency test when the mixer is charged and operated in accordance with practice which may be established as routine for the job.

The mixing period should be measured from the time when all the solid materials are in the mixer drum provided that all the mixing water shall be introduced before 25 per cent of the mixing time has elapsed. Over-mixing of more than three times the above required periods should not be permitted and for that reason all mixing equipment should be so arranged that it can be stopped and started under full load.

A batch timer and counter, including an automatic lock which will release the discharge lever only at the end of the proper mixing period, and an audible indicator, should be installed and maintained in operation on each concrete mixer. On truck mixers the audible indicator is unnecessary. For jobs requiring 2 cu. yd. or larger stationary mixers a recording watt meter or consistency meter has definite advantages and should be required on each mixer because it is more reliable as a batch counter, and because it provides a permanent record of number of batches, type of batch, mixing time, retempering, and the uniformity of consistency. Failure of any of this equipment should be sufficient cause for discontinuance of the use of the mixer until the device is repaired or a new one is installed.

Retempering

Retempering or indiscriminate addition of water to delayed batches which have become stiffer than the usual consistency should be pro-

hibited. Under careful supervision an increment of water may sometimes be added with results beneficial to essential workability, provided the maximum allowable water-cement ratio is not exceeded. Concrete delayed in placing should not be wasted unless the amount of additional water, necessary to restore sufficient workability to permit it to be placed, would exceed the allowable water-cement ratio. Batches considerably wetter than the specified consistency, or batches in which there has been some obvious error in the measurement of solid ingredients, particularly a shortage of cement, should be wasted unless proper corrections can be made at once and the corrected batch thoroughly mixed.

Ready mixed concrete

The use of ready mixed concrete is a relatively new and important development in connection with general concrete construction—particularly in metropolitan areas. A ready mixed concrete operation, adequately equipped and supervised, affords excellent opportunities for the control of concrete quality. Thus, ready mixed concrete may be made to offer the small job the control facilities ordinarily available only to large jobs.

Ready mixed concrete may be centrally mixed, truck mixed (mixed in transit), or partially centrally mixed and partially truck mixed (often referred to as "shrink-mixing"). The use of ready mixed concrete introduces certain new problems of control to which, while not fundamentally different from those encountered in other forms of concreting operations, attention should be directed.

Centrally mixed concrete presents about the same problems as job-mixed concrete which must be transported for considerable distances from the plant to the place of deposit. The possibilities of loss in consistency, resulting from lapse of time between mixing and placing, should be given consideration. For normal periods and temperature, and when agitator trucks are used for transportation, this factor is generally negligible. Nevertheless, precautions should be taken to minimize loss of slump by expediting delivery, eliminating delays and, in warm weather, keeping the concrete as cool as practicable by using cold mixing water, avoiding hot cement, and by shading and sprinkling aggregates. Under conditions where loss of slump is likely to be a serious matter, on work in areas not served by established central mixing plants where these precautions may be taken to an effective degree, mixing at the forms will eliminate the difficulty.

In the case of truck mixer operations particular care should be exercised in the control of the consistency since, under normal operat-

ing conditions a considerable quantity of concrete arrives at the job before there is opportunity to make adjustments. Variations in consistency may be minimized by:

(1) Handling aggregates in such a manner as to reduce variations in grading and moisture content to the greatest practicable extent.

(2) Exercising accurate control over the quantity of mixing water which may be admitted to the truck mixer.

(3) Telephone communication between the point of placing and the batching plant.

(4) Responsible technical supervision of the entire operation.

While every effort should be made to have concrete batches arrive at the job at a suitable consistency, it is possible to make adjustments after arrival if proper provisions are made in specifications and on the job. If the batch arrives too dry, but containing less water than the maximum allowed, additional mixing water, within specification limits, may be added and additional mixing done at the job site. In general, batches which arrive too dry, and already containing the maximum allowable mixing water, or batches which arrive too wet, should be rejected. However, where conditions permit, adjustments may be made in the first instance by the addition of proportionate quantities of water and cement, and in the second instance, by the addition of proportionate quantities of cement and aggregate. If the proportioning plant is nearby, the truck mixer may be returned to it for the necessary adjustments, or these adjustments may be made at job site. However, such adjustments are to be avoided, since they interfere with orderly routine, and every care should be exercised to see that concrete arrives at the job with the desired consistency. For general construction work outside of areas served by well supervised ready-mix operators, best results are more likely of attainment by the use of mixers stationed at the forms. Only under conditions known to be definitely favorable to consistency control and minimum slump loss, will truck mixing approach the uniformity of results at minimum water content obtainable under similar conditions by mixing at the forms.

Truck mixers should be operated at a mixing speed in accordance with the recommendations of their manufacturer and should also be capable of operating at a suitable lower agitating speed of not less than 2 nor more than 6 r.p.m. of the drum.

The time of mixing in truck mixers should be as follows: for top-door-loading mixers when all aggregates are charged simultaneously, for end-loading mixers when the mixed concrete does not exceed 50 per cent of the gross drum volume, and for shrink mixing, not less

than 40 revolutions of the truck mixer drum at mixing speed; for top-door-loading mixers when fine and coarse aggregates are charged separately into the drum, and for end-loading mixers when the mixed concrete is in excess of 50 per cent of the gross volume of the mixer drum, not less than 60 revolutions of the truck mixer drum at mixing speed. If, from any cause, this amount of mixing is insufficient, as determined by the subsequent tests for mixer efficiency or by other means, additional mixing should be required until satisfactory results are obtained. In truck mixing and shrink mixing, not more than 150 revolutions of the drum should be at a peripheral speed in excess of that designated as agitator speed. Additional mixing, if necessary, should be at agitator speed.

Measure of mixing efficiency

The efficiency of the mixing operation in all types of mixers should be such that, at the end of the prescribed or permitted mixing period, the S/C (sand-cement) and the W/C (water-cement) ratio of the mortar in any part of the batch is within 10 per cent of that in any other part. This should be tested by taking samples from three well separated localities in the mixer, and testing them by means of the U. S. Bureau of Reclamation "Mixer Efficiency Test Procedure," or the Iowa State College method for the "Field Determination of the Constituents of Fresh Concrete." The ratio of coarse aggregate to mortar should appear to be uniform in all parts of the mixer on visual inspection and this should be borne out by subsequent observation of the workability of the concrete.

The Discharging Operation

No portion of the time required for discharging any type of mixer should be considered a part of the required net mixing time. The discharge facilities of all types of mixers should be capable of ready discharge of concrete of the stiffest consistencies which may be placed by means of vibration. To meet this requirement, all mixers should be capable of ready discharge of concrete of 1 in. slump.

To preserve the uniformity of distribution of materials and the usual homogenity of the concrete in the mixer immediately prior to discharge, all types of mixers discharging into hoppers, buckets, cars, etc., should be so equipped that the concrete will drop vertically, not diagonally, into such containers in accordance with the principles shown in the sketches which constitute a part of these recommendations. Separation of coarse aggregate from the mortar, commonly resulting as concrete is discharged from the mixer, is thus avoided.

The blade arrangement and discharge mechanism of all types of mixers, including agitating, shrink, and truck mixers, should be such

that the additional percentage of coarse aggregate in the last 10 per cent of concrete discharged will not exceed the percentage of coarse aggregate in the first 10 per cent of concrete discharged by more than 20 per cent. This may be readily determined by comparing the weights of coarse aggregate retained when 100 lb. of concrete from each end of the batch is washed over a ¾ in. screen. Until or unless equipment is available which will meet this requirement, and is operated in this manner, the fraction of each batch found to contain excessive amounts of coarse aggregate should be withheld from discharge and mixed with the succeeding batch. As heretofore recommended in this case, the size of batches charged into the mixer should be reduced by this amount. In some truck mixers this type of separation may be avoided by reversing the direction of rotation for one or two revolutions prior to the final discharge.

General

In general, all of the foregoing recommendations should be considered where applicable in connection with the mixers of proper capacity for the job regardless of the size of the job or its classification.

PLACING

Essential Requirements

Only those methods and arrangements of equipment should be used which will reduce to a minimum the separation of coarse aggregate from the mortar at all points from the mixer to the forms. For highest quality of concrete *in place*, the placing equipment should be considered for its ability to handle mixes ideal for the particular work, from the standpoint of the concrete placeability and workability *in the forms* (preferably by means of vibration where this is applicable). Equipment should be capable of expeditious placing of concrete of the proper consistency, grading and maximum size of aggregate, at the rate most advantageous to good workmanship. Selection or approval of any part of placing equipment requiring, for efficient operation, mixes containing larger proportions of sand, cement, or water, or smaller coarse aggregates, than indicated for best results on the above basis, is not recommended.

Avoidance of Separation

The most important consideration in handling and placing concrete is the avoidance of separation of coarse aggregate from the concrete. Particular attention must be paid to that tendency at ends of chutes and conveyor belts, at hopper gates and at all other points of discharge, if uniformity and homogeneity of concrete in place and good workmanship are to be assured. No consideration should be given

to the common fallacy that separation occurring in handling will be eliminated in the course of other operations. Separation should be prevented—not corrected after its occurrence.

Important in arranging equipment to prevent separation is the provision that the concrete shall *drop vertically* into the center of whatever container receives it. The importance of this increases very greatly with increases in slumps, maximum size, amounts of coarse aggregate, and with reductions in cement content. Falling concrete should be closely confined in a down pipe of the proper size to within a few feet of the place of deposit in the forms or other container *and the final drop must be vertical* if separation is to be corrected by prompt reassembly of the ingredients as they fall. Application of this principle to various common details of concrete handling equipment is shown in the accompanying sketches.

Where applicable, the use of bottom discharge buckets is a superior method of handling and placing mass concrete provided: (1) they are of a size, and may be discharged in a manner and with such frequency, that the concrete may be placed in approximately horizontal layers while the previous layer is still soft; (2) they are capable of discharging concrete of the stiffest consistency specified; (3) successive batches are so placed as to afford opportunity for thoroughly working the concrete by means of internal vibrators; (4) complete mixer batches only are placed in the bucket; and (5) separation is avoided in filling the buckets (see sketch).

Concrete should not be dropped through reinforcement steel, or into any deep form whether reinforcement is present or not, so as to cause separation of the coarse aggregate from the mortar by repeatedly hitting rods or the sides of the form as it falls. For placing under such conditions, hoppers and, if possible, vertical ducts should be used in the forms, or other means employed so that the concrete may reach the place of final deposit without separation or coating the steel and forms with mortar that will dry out long before it can be covered with concrete. In difficult cases of deposit of this kind in deep, narrow, reinforced walls, where even narrow rectangular ducts can not be inserted, good results may be obtained by closing each third space between studs on one side of the wall for a duct through which to drop the concrete which then enters the form through holes cut in the sheathing at vertical intervals not greater than 4 ft. as the concrete rises to each opening. A pocket should be provided at the bottom of each duct successively, below each opening, so that the concrete will stop and flow easily over into the forms with minimum scattering and separation. A good internal vibrator should be in

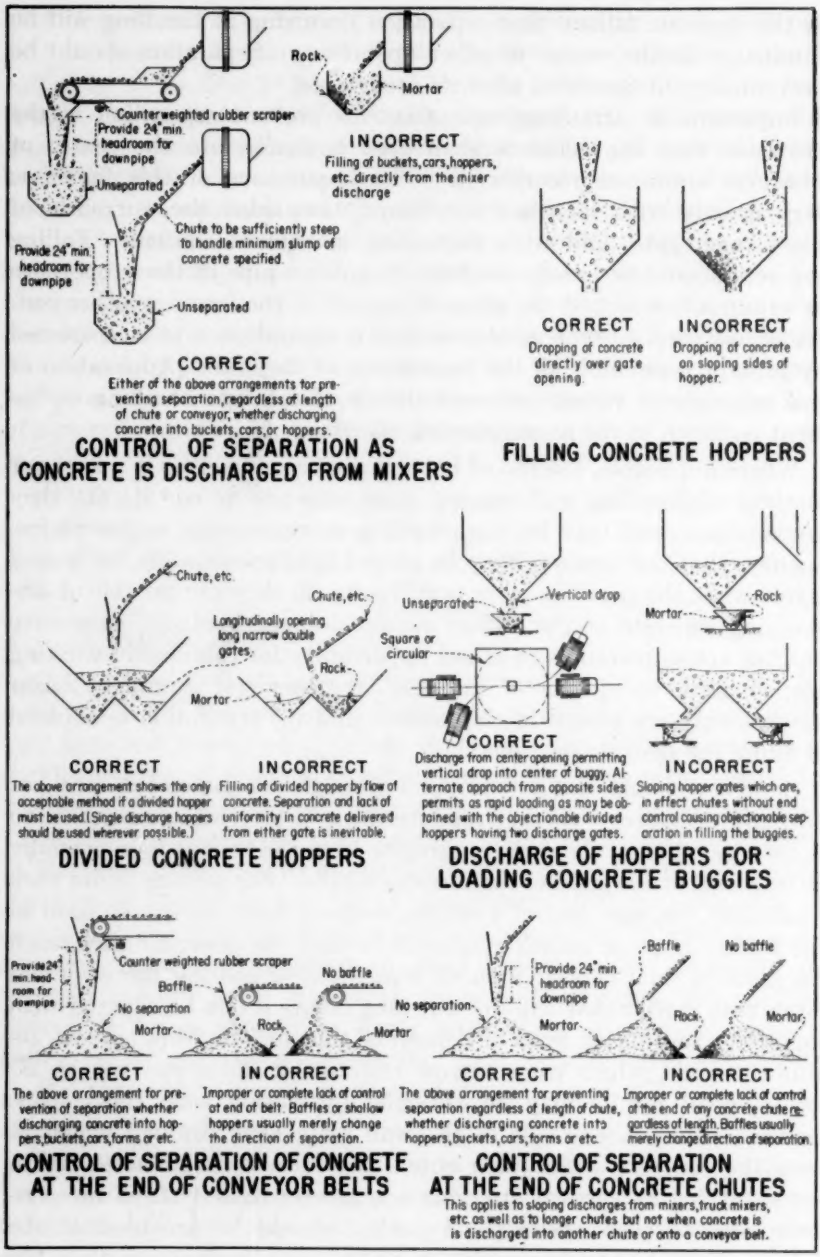
Rock
Mortar
Counterweighted rubber scraper
Provide 24" min. headroom for downpipe
Unseparated
INCORRECT
Filling of buckets, cars, hoppers, etc. directly from the mixer discharge
Chute to be sufficiently steep to handle minimum slump of concrete specified.
Provide 24" min. headroom for downpipe
Unseparated
CORRECT
Either of the above arrangements for preventing separation, regardless of length of chute or conveyor, whether discharging concrete into buckets, cars, or hoppers.
CONTROL OF SEPARATION AS CONCRETE IS DISCHARGED FROM MIXERS
CORRECT
Dropping of concrete directly over gate opening.
INCORRECT
Dropping of concrete on sloping sides of hopper.
FILLING CONCRETE HOPPERS
Chute, etc.
Chute, etc.
Longitudinally opening long narrow double gates.
Rock
Mortar
CORRECT
The above arrangement shows the only acceptable method if a divided hopper must be used. (Single discharge hoppers should be used wherever possible.)
INCORRECT
Filling of divided hopper by flow of concrete. Separation and lack of uniformity in concrete delivered from either gate is inevitable.
DIVIDED CONCRETE HOPPERS
Unseparated
Vertical drop
Square or circular
CORRECT
Discharge from center opening permitting vertical drop into center of buggy. Alternate approach from opposite sides permits as rapid loading as may be obtained with the objectionable divided hoppers having two discharge gates.
Rock
Mortar
INCORRECT
Sloping hopper gates which are, in effect chutes without end control causing objectionable separation in filling the buggies.
DISCHARGE OF HOPPERS FOR LOADING CONCRETE BUGGIES
Provide 24" min. headroom for downpipe
Counter weighted rubber scraper
Baffle
No separation
Mortar
CORRECT
The above arrangement for prevention of separation whether discharging concrete into hoppers, buckets, cars, forms or etc.
No baffle
Rock
Mortar
INCORRECT
Improper or complete lack of control at end of belt. Baffles or shallow hoppers usually merely change the direction of separation.
CONTROL OF SEPARATION OF CONCRETE AT THE END OF CONVEYOR BELTS
Provide 24" min. headroom for downpipe
No separation
Mortar
CORRECT
The above arrangement for preventing separation regardless of length of chute, whether discharging concrete into hoppers, buckets, cars, forms or etc.
Baffle
No baffle
Rock
Mortar
INCORRECT
Improper or complete lack of control at the end of any concrete chute regardless of length. Baffles usually merely change direction of separation.
CONTROL OF SEPARATION AT THE END OF CONCRETE CHUTES
This applies to sloping discharges from mixers, truck mixers, etc. as well as to longer chutes but not when concrete is discharged into another chute or onto a conveyor belt.

FIG. 2—CORRECT AND INCORRECT METHODS OF HANDLING MIXED CONCRETE

operation on each side of an opening while concrete is entering the forms.

Vibration

Except for a few special sections, such as thin slabs, no method of compaction and consolidation of concrete in place is superior to effective internal vibration. Vibration contributes general improvement to concrete work in the fact that superior workmanship may be obtained at lower cost, and because it permits successful routine use of a concrete less wet than the usual "medium" consistency, resulting in more durable, higher quality concrete. The advantages and importance of vibration where applicable are now so well established that it is worthy of serious favorable consideration for small jobs of a few hundred yards, and its requirement is recommended for all larger concrete work.

Detailed recommendations are omitted here and the reader is referred to "Recommendations for Placing Concrete by Vibration" reported by A. C. I. Committee 609. Sufficient to say that the name of vibration is not enough. Equipment should be powerful, of high frequency, efficient, and rugged. Operators should be experienced, competent, dependable and energetic. Ample standby units and parts should be provided. Care should be exercised that concrete is not over-vibrated, particularly if it exceeds 4 in. in slump. Vibrators should not be used to "transport" concrete in the forms. Vibrators should be inserted and withdrawn at many points, from 18 in. to 30 in. apart, for short periods, (usually from 5 to 15 seconds is sufficient) in preference to insertion for longer periods at wider intervals. Systematic spacing of insertions of the vibrator should be established to insure that no concrete remains unvibrated.

Inadvertent or intended revibration of concrete or steel embedded in it, any time before the concrete becomes so far set that continued revibration will not "melt" it together again, is not detrimental but may actually increase the strength of the concrete and its bond with the steel. As far as the ordinary job is concerned there is little likelihood of damage from revibration of lower lifts or from vibration transmitted by the steel, provided there is avoidance of actual rupture of newly hardened concrete that can not be "healed" by the vibration.

Provision for Handling Concrete of the Proper Consistency

Objection is frequently made on the job to a relative stiff consistency which will not flow down a certain chute, drop out of a certain hopper or discharge through certain gates, although it is freely admitted

that it is readily workable in the forms, particularly when properly vibrated. Such an objection is not valid and should not be sustained if the drier consistency has been determined in advance to be of practicable workability in the forms and has been made a requirement of the specifications. It is the function of concrete handling and placing equipment to handle and place concrete of proportions and consistencies that can be properly worked in the forms and selected for the results required in the forms. Limitations on consistencies and proportions should not be imposed by inadequate chutes, hoppers or gates.

Thus it is important, in the design and approval of concrete handling equipment that chutes, where necessary, be amply steep, metal or metal lined, round bottomed, of large size, rigid and protected from overflow. Discharge gates and hoppers should be large enough to pass quickly and freely concrete of the stiffest consistency likely to be found practicable for placing in the forms by means of vibration. In many cases chutes steeper than 2 to 1 should be used, and double or triple the usual area of hopper outlets and gates would not be excessive.

Except where loss of slump due to delayed placing is unimportant and no increase in the original amount of mixing water is made because of it, or when remixing without additional water after a period of delay is desirable and practicable for reduction of shrinkage due to setting, every effort should be made to keep as short as possible the time elapsed from the moment water and cement come together, and the arrival of the concrete in the forms. This facilitates control of uniformity of consistency of the concrete in the forms, and reduces to a minimum the water content of the concrete, and the variable loss of slump between the mixer and the forms. For this same reason concrete should not be exposed in thin streams in long chutes or on long conveyor belts in which condition it is subject to the variable effects of all kinds of weather. It should not be necessary to mix the concrete any wetter than it may be worked in the forms by means of vibration.

GENERAL CONSIDERATIONS

In mixing and placing concrete, all concerned should remain aware that shrinkage cracks and lack of durability are primarily proportional to the volume of mixing water per unit volume of concrete within the range of practical and suitable mixes for most work. Though in many cases they are, they *may* not be correlative to the water-cement ratio, except for mixes otherwise identical in aggregate

and cement content, or for mixes identical only in water content and consistency. An example is the case in which both cement and water are added without changing the water-cement ratio but with an increase in slump. Another, when increases in cement and water are accompanied by use of finer grading of aggregate resulting in little change in consistency. Such variations in unit water content proportionately affect the ultimate quality of the concrete without relation to the water-cement ratio. The more a form is filled with the right combination of solids, and the less it is filled with water, the better will be the resulting concrete. Thus to this end, moderation in the use of water, cement, and fine aggregate, and use of aggregate graded to the largest practical maximum size, all in conformity with the elemental needs of the work, should be consistently practiced. Only as much cement should be used as is required to obtain adequate strength and other essential properties; and only sufficient water and fine aggregate should be used as is required to obtain no more than a degree of workability which may be properly worked in the forms by means of vibration or by more appropriate methods where vibration is not applicable.

Sufficient mixing and placing capacity should be provided so that the work may be kept alive and free (with judgment in planning the progress of the work) from "cold joints." In all formed structures, including tunnel linings, concrete should be placed in horizontal layers of no greater depth than 2 ft., particularly avoiding inclined layers and inclined construction joints. It is very important that each layer be placed while the previous layer is still soft. Concrete should not be allowed or caused to flow horizontally or on slopes in the forms. Where concrete must be placed on a slight slope, placing should begin at the lower end of the slope and progress upward thereby increasing the compaction of the concrete. Pneumatic means of placing concrete should, in general, be avoided wherever an alternate method may be used.

Construction Joints

For the sake of appearance, irregular construction joints should not be permitted. Each lift of concrete should be filled up to a temporary grade strip, or preferably a "V" or a slightly beveled rectangular strip should be left on the forms at the line of the joint where such grooves can be located so as not to detract from the appearance of the work and so as to be less unsightly than the usual construction joint. Particular precautions should be taken to secure the forms tightly against the concrete at the joint, since, otherwise,

an unsightly offset will occur and mortar from the subsequently placed concrete will disfigure the surface of the concrete in place.

The surface of the construction joint should be prepared in a manner that will insure bonding with the concrete later placed on it, and if the joint must be water tight the preparation of the joint must be particularly thorough; in fact, in that case it may be worth while to take the added precaution of embedding a metal water stop strip so as to span the joint. Where laitance is present or the wetter consistencies of concrete have been used, the initial preparation should be the removal of all laitance, surface film, and mortar from protruding coarse aggregate by means of a strong jet of air and water at approximately 100 p.s.i. pressure after the concrete has hardened sufficiently to prevent the jet from raveling the surface below the desired depth and from forming cloudy pools of water that will leave a film on the surface when they dry. No initial cleanup preparation is necessary on horizontal joints in mass concrete that has been placed at dry consistencies with the aid of vibrators, if the surface concrete is protected from excess working due to keyway construction, endeavor to embed all coarse aggregate, and general job traffic, until it has set. The joint surface should be well moist cured preferably by a 1-inch layer of saturated sand, and, if possible, it should never be permitted to dry out during the interval before concrete is placed upon it, at which time it should be restored to the bright clean condition existing immediately following the initial jetting by means of vigorous scrubbing with fine wire hand brushes or by sand blasting as necessary. If the surface has been properly sand cured, very little final scrubbing or sand blasting will be necessary. In cases where jetting and brushing is too difficult or interferes with other operations, and provided there is no deep hardened laitance on the joint, the joint may be adequately prepared for good water tight bonding by thorough removal of the surface film and dirt by sand blasting immediately prior to placing the new concrete. In all cases the new concrete should be preceded by about one inch of soft mortar of the same proportions as that in the concrete. When accessible this should be scrubbed into the surface of the joint with wire brooms.

Finishing of Unformed Surfaces

For the most durable results in any type of finished, unformed concrete surfaces, the following general requirements should be made. Concrete of the stiffest consistency that can be properly

placed and vibrated should be used. In the initial operations of screeding, floating, and first troweling, the surface of the concrete should be worked *as little as possible* in obtaining the desired result. Each step in the finishing operation from the secondary floating to the final operation should be delayed *as long as possible* and yet permit the desired result to be obtained. The use of any finishing tool in any area where water has accumulated should be prohibited. Dry topping and mortar topping should be avoided. The surface of the concrete should be directly finished to the texture desired.

Cold Weather Concreting

For concrete in sections thinner than would be called mass concrete, when the weather is such that freezing of the concrete may occur, arrangements should be made to heat the mixing water and the aggregate so that the temperature of the fresh concrete in place is approximately 70°F. At this temperature setting commences at once and, due to the heat liberated by hydration, will progress at a normal rate, provided the work is protected for several days from dissipation of this heat more rapidly than it is created. This procedure is eminently preferable to that of placing the concrete cold at any temperature above freezing and then endeavoring to heat it in place by salamanders or other means; however, under severe conditions it may be necessary to surround the work with some form of heating protection in addition to protection by insulation. In connection with cold weather protection, such precautions should be taken that the curing of the concrete will not be impaired and that no portions of the work will become overheated. It is important that there should be a curing period of sufficient length at temperatures above freezing so that when it is exposed to temperatures below freezing at the end of the curing period the concrete will not be injured.

With adequate protection of the surfaces from freezing, the minimum temperature of freshly mixed mass concrete may be permitted to be as low as 40°F when placed, because the heat of hydration is lost much more slowly from this type of concrete.

Hot Weather Concreting

For best ultimate quality, concrete should be placed at the lowest practicable temperature during hot weather. Effective results can usually be accomplished by concreting only at night, sprinkling or cooling the aggregate, avoiding hot cement, and using very cold mixing water. Any combination or degree of these practices that may be feasible for the particular work are important and should be

required. Curing should preferably be obtained by sprinkling or covering with moist burlap for its additional cooling value. If curing must be done by means of black bituminous sealing compounds, they should be given a coat of whitewash promptly so as not to expose the heat absorbing black surface to the sun.

Forms

Provisions as to certain types of forms are necessary because, when they are not followed, concrete containing excessive amounts of sand, cement, or water is usually necessary in order to obtain acceptable workmanship. Horizontally moving slip forms should not be used on slab work on which standard types of paving and finishing machines can be used. Preferably such machines should be equipped with an effective form of vibration or mechanical device for working and compacting the concrete. A continually moving slip form, however, should be used in preference to fixed forms whenever possible on slope paving slabs or steeply curved inverts.

Workmanship in placing concrete is largely judged by the appearance of the work on removal of the forms, and appearance in itself is an important quality which must usually be obtained when the concrete is placed. It is therefore necessary in these recommendations to mention other requirements for satisfactory results.

With the general adoption of vibration of concrete, forms must not only be built substantially, but they must be tight, since, otherwise, unsightly sand streaks and gravel pockets will be caused by loss of mortar made unusually liquid during vibration. Exceptional care should be exercised to insure that all form panel joints, corners, and connections and all seams between all types of sheathing are completely tight immediately prior to placing the concrete. Because of their superior tightness where they penetrate the forms, tie rods are recommended in preference to tie wires. Where wires must be used holes for them through the forms should be as small as possible to keep mortar leakage at a minimum.

Forms should be protected from deterioration, weather and shrinkage prior to concreting by proper oiling or by effective wetting. Form surfaces should be clean and of uniform texture. Where they are permissible, re-used forms should be carefully cleaned and oiled. Steel forms should be thoroughly cleaned but never sand blasted nor abraded to bright metal. Where "peeling" is encountered with steel forms, leaving the cleaned, oiled, forms in the sun for a day, or vigorously rubbing the affected areas with liquid paraffin will usually improve the condition. It is generally less expensive and more

satisfactory to obtain the desired surface effect by proper treatment and preparation of the forms than it is to obtain it by working over the concrete after the forms are removed.

CLOSURE

In general, all of the foregoing features should be considered, insofar as applicable, in connection with each job classification, regardless of type or capacity of equipment.

This report, open for discussion at the 36th Annual Convention and to subsequent discussion which should reach the Institute Secretary in triplicate by Apr. 1, 1940 will be considered by Committee 614 in the perfection of the report for later proposed adoption as a Standard Recommended Practice of the Institute. Discussion is scheduled for publication in the Supplement ***to be issued with the September 1940*** JOURNAL.

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Discussion of Report of Committee 614, Lewis H. Tuthill, Chairman

Proposed Recommended Practice for Measuring, Mixing, and Placing Concrete*

BY W. F. KELLERMANN, ARTHUR A. LEVISON, LEWIS H. TUTHILL, HARRY C. SHIELDS, DUFF A. ABRAMS AND CLOSURE BY THE COMMITTEE

CONVENTION DISCUSSION

W. F. Kellermann†—While it is more or less a universal practice to overload mixers, that is, beyond capacity stamped on the frame, the question frequently arises whether or not to allow a ten per cent overload. There are manufacturers of mixing equipment in the room in addition to members of the committee, and I was wondering whether or not we could not forestall some of this argument in the future by stating right on the mixer the quantity of concrete it will mix. Then we won't have an argument as to whether we shall allow one per cent or ten per cent or twenty-five per cent overload.

Arthur A. Levison‡—I have two brief comments relative to this report. The first has to do with recommended practice with bins and batchers for porportioning the materials for concrete. I think we will all recognize that the report sets up the best practice for proportioning operations for the ingredients of concrete. In job practice we are frequently confounded by the conditions in specifications written for the individual job. I am making the plea that the specification writers, as well as the contractors, pay particular attention to the report of Committee 614 and try to take therefrom proper guidance in the establishment and interpretation of specifications for a given job. To illustrate the point a little more definitely, some of the larger engineer-

*JOURNAL, Amer. Concrete Inst., Feb. 1940.
†U. S. Public Roads Administration, Washington, D. C.
‡Blaw-Knox Co., Pittsburgh, and member committee 614.

ing organizations in this country have established bat hing specifications calling for separate sizes of aggregate, automatic batching of the aggregate, cement and water, and other requirements which are entirely right and desirable for large projects. In any such project the plant remains on the site for a considerable time, frequently two years or more. We often find that the same specifications, or with little variation, are applied to the smaller project. There is also a problem for contractors who bid the work of making proper evaluation of these specifications in terms of the cost of such equipment per cubic yard of concrete for the yardage involved in the work. I think that following the recommended practice of Committee 614, with proper gauge of what the size of project permits, a good deal can be accomplished toward lowering the cost per cubic yard.

The other item I have in mind is in connection with that portion of the report which has to do with the use of truck mixers for concrete delivered from a central charging plant to the forms at some distance. It is recognized that under some conditions, there might be an appreciable loss of slump between the charging plant and the point of delivery, and that this may be a troublesome item. The addition of water to offset the loss of slump requires the addition of cement to maintain the water-cement ratio and involves additional expense. In addition to that, this loss of slump enroute is not always uniform. When the concrete arrives at the job, lack of uniformity prevails in the consistency of the delivered concrete. So I want to ask Mr. Tuthill, in view of his personal experience with work carried on under these conditions, whether the use of truck mixed concrete could be helped or improved by not adding water at the charging plant, but simply charging the aggregate and cement into the trucks, having the truck mixer equipped with an adequate water measuring tank and adding the water after the truck mixer has arrived at destination? It may be that such a method of operation would obviate the loss of slump and be helpful in controlling the consistency of the concrete from load to load. In other words, if a given truck comes up to the point of deposit of the concrete and the prescribed amount of water is added—an amount to produce required slump, and still be the consistency proves to be unsatisfactory, correction by adding all of the mixing water at the job, could be made on succeeding truck loads.

Mr. Tuthill—The answer is that the results with truck mixers would be improved, but, for large work outside urban areas, where there is plenty of working space, I would rather spend the extra money that a fleet of such trucks would cost over ordinary batch trucks and a mixer at the forms, for some of the other improvements recommended by

Committee 614. Another thing involved in the suggestion made by Mr. Levison is the time consumed for mixing after the trucks arrive at the forms. If specifications were rigidly set up on the suggested basis, you might accomplish the desired result, but, if specifications have not been set up on that basis, and the contractor comes on the job with a fleet of truck mixers and is permitted to use them, I can assure you there would be no time lost in mixing at the forms.

*Harry C. Shields**—In the suggestion offered by Mr. Levison, I see some advantages coupled with much danger. If the ready-mixed concrete operator, with a fleet of trucks, servicing a job on the basis suggested, permitted each truck mixer operator to charge the water into the mix—at that point it would cease to be a controlled batch of concrete so far as the ready-mixed concrete manufacturer is concerned. With fifteen truck mixers operating, you will have 15 truck drivers adding water to the concrete that the producers must stand back of and rise or fall by. It has been my personal experience in the field, and not infrequently, to encounter conditions in which a faithful performance of the delivery of a given quality of concrete on the job, is interrupted or prevented by some foreman who says, "Charlie, this is a little bit too stiff, give her another shot of water." It does happen.

Mr. Tuthill—I do not want any one to gain the impression that there is an intention or implication in the report to the effect that ready-mixed concrete as described by the speakers during this session, is not approved by the committee; or, as far as that is concerned, by me personally, because I am in very complete and hearty accord with all that has been expressed in these other two papers.†

Mr. Levison‡—After reading Mr. Tuthill's reply to my discussion and the comments of Harry C. Shields, I would like to submit additional information which has a bearing on this matter of controlling truck mixed concrete.

The practice of prohibiting the addition of water to the batch in the drum of a truck mixer until arrival of the unit at the job, is in effect, in a number of localities, notably on much of the work in Greater New York. While it is true that this involves a delay after arrival of the truck mixer at the job site, the truck mixer operators apparently are still anxious to compete for the business against job mixed concrete. That, to my mind, is a matter of economics and competition which the supplier has to work out.

Now, as to the matter of controlling the water that is admitted to the concrete, these specifications that prohibit adding the water

*Marquette Cement Manufacturing Co., Chicago.
†One by Foster, Knopel & Whitten, February JOURNAL; the other by H. F. Thomson, April JOURNAL.
‡Supplemental discussion.

until the truck mixer arrives at the job use this means for inspecting and controlling the amount of mixing water. While it is true that the truck mixer driver may admit the water to the truck mixer drum, he does it under close inspection with the truck mixer standing still. There is no opportunity for truck mixer drivers on their own initiative to control the amount of mixing water and thus the quality of the concrete.

I appreciate that if the specifications are not set up in advance to provide for this type of operation, it would be difficult to enforce them after the jobs get under way, unless the engineers in charge took the position that this was the only practical way for controlling the uniformity of consistency from batch to batch.

Again viewing this method of adding mixing water at the job from the economic point of view, it must be admitted that under this system it would take more truck mixers to supply a given yardage per hour or per day than where the water is put into the truck mixer drum at the charging plant, or enroute to the job. However, there are many places where this is required and we have certain kinds of work in the Pittsburgh area where the ready-mix company that is furnishing the concrete, or the contractor who owns the truck mixers, selects the truck mixer method for supplying the concrete in competition with other established methods.

BY DUFF A. ABRAMS*

(*1*) *Effect of Quantity Control on Uniformity of Concrete*

The writer is forced to the conclusion that the Committee has placed too much emphasis on the control of quantities and the mixing of concrete. The reasons for this conclusion will be developed in sections (1) and (2) of this discussion.

Grand Coulee Dam now nearing completion by the U. S. Bureau of Reclamation contains 10½ million cu. yd. of concrete. Millions of dollars have been spent in blending cements, washing, screening, classifying and draining aggregates, tests and inspection of concrete materials and concreting equipment, accurate batching and unusually thorough mixing. The sand is separated into 3 sizes which are recombined to a fixed grading, gravel is separated into 4 sizes, each concrete material is weighed to 5 lb., consistency meters control each batch, each mixer is subjected to efficiency tests 3 times daily. This is no doubt the most complete and accurate control of quantities and mixing that has ever been exercised on any job in the history of concrete. In spite of unprecedented control we find that the 28 day concrete

*Consulting Engineer, New York.

strengths in Sept. 1937, showed a spread of 70 per cent between the low and high values (3750 to 6400, average 5395 p.s.i., O. G. Patch, JOURNAL, Jan. 1939); in July 1937, the spread was 50 per cent (Patch, A. C. I. JOURNAL, June, 1939, Table 3). It should be noted that the strength tests thus far reported were made during months which would be expected to show minimum variations in temperatures and other disturbing factors.

Great accuracy in control of quantities is possible, yet we must face realities to the extent of recognizing that unprecedented control of quantities on this job, even when combined with unusual thoroughness of mixing, accomplished nothing toward producing *uniform concrete.* Other jobs where little or no control was attempted have shown equal or better concrete uniformity as measured by compression tests.

(2) *Measure of Concrete Mixer Efficiency*

The report before us states: "Thorough mixing is, of course, a first essential," hence the Committee probably considered the time of mixing concrete its most important recommendation. The mixing time required is dependent on mixer tests (p. 341):

> The efficiency of the mixing operation in all types of mixers should be such that, at the end of the prescribed or permitted mixing period, the S/C (sand-cement) and the W/C (water-cement) ratio of the mortar in any part of the batch is within 10 per cent of that in any other part. This should be tested by taking samples from three well separated localities in the mixer, and testing them by means of the U. S. Bureau of Reclamation "Mixer Efficienty Test Procedure," or the Iowa State College method for the "Field Determination of the Constituents of Fresh Concrete.

The two methods mentioned are not comparable; they give quite dissimilar results. The U.S.B.R. method comes first, hence I assume that it is preferred by the Committee. However there is a greater difficulty, since I know of no document sponsored by the U.S.B.R. under the title "Mixer Efficiency Test Procedure." I assume that this is the "Mortar Test for Mixer Efficiency" which appeared in the "Concrete Manual," 1938, revised edition 1939, and the method which was described by Mr. Patch under the title "Mixer Efficiency or Mortar Mix Test," (A. C. I. JOURNAL, Jan. 1939).

My discussion of Mr. Patch's paper (JOURNAL, June and Sept., Suppl., 1939) pointed out that:

(a) The U.S.B.R. method misinterprets useful mixer data, since calculations of efficiency are based on a fallacy as a result of an error in elementary arithmetic.

(b) The computed quantities of sand, cement and water were known to be erroneous, hence both S/C and W/C were obtained by dividing one wrong number by another wrong number.

(c) The mixer errors obtained by this method were about 5 times the mean variation of the quantities of solids below the 100 sieve in the 3 samples from the batches reported.

(d) All mixer efficiency figures obtained by this method (such as 102, 110, etc.) are subject to a variation of 4 points *in either direction*, due to unavoidable errors in the test for the quantity of 0 - 100 sand in the mix.

(e) In using this test at Grand Coulee Dam, the U.S.B.R. held the average mixer error to the unreasonable refinement of about 1 per cent when measured by the mean variation of three samples from the batch, a variation which included all accidental errors of sampling and testing. This should be contrasted with a spread of 70 per cent in the strength of concrete in a single month.

(f) That Messrs. Patch and Ruettgers of the U.S.B.R. refused to divulge how mortar samples were secured in the mixer tests at Grand Coulee Dam on which the method was based.

Someone who encounters this method in the JOURNAL might attempt to use it. Even in the hands of men of the widest experience, the method gives unreliable and in many instances absurd results; for example, Mr. Patch stated that after securing a mortar sample by wet-screening, both the mortar and the remaining concrete showed a *higher* W/C than the original batch (JOURNAL, Jan. 1939).

Mr. Patch also showed (JOURNAL, June, 1939, Table 10) that four 4-yd. mixers at Grand Coulee Dam (one set of tests on each mixer during each of 3 daily shifts for 1 week) gave average errors as follows:

Shift	Hours	Mixer Errors	
		%	Relative
Night	11–7	2.4	100
Day	7–3	3.5	145
Swing	3–11	4.5	188

The error of the *same mixers* in the day shift was 45 per cent higher and in the swing shift 88 per cent higher than the night shift. One might conclude from these tests that a dime's worth of black paint applied to the windows of the mixer building would have a most important effect in improving mixer efficiency.

The method of securing samples is the crux of the U.S.B.R. test. Until the method of securing mortar samples from the mixer and the identity of the U.S.B.R. "test procedure" are clarified, this section of the Report will remain inoperative. If the "Manual" method is the one intended, it should be discarded or revised so that the computed efficiencies have some recognizable relation to the variations that actually exist in the batch. If this is not the method intended, we must conclude that the Committee is sponsoring a mixer test which is not known to the public.

(*3*) *Report Goes Beyond "Best Practice"*

In recommending metal water-stops the Committee has overstepped its field and entered the province of design. If such water-

stops are necessary for water-tight joints, why did the U. S. B. R. omit them on the horizontal joints of Hoover and Grand Coulee Dams where heads up to 725 ft. are encountered?

The Committee restricts its recommendations to wet sand curing; there are, of course, other well-recognized and efficient methods of curing concrete.

(*4*) *Cleanup of Horizontal Construction Joints*

It seems doubtful whether size and grading of sand, curing, or cleanup of joints fall within the scope of this Committee; however since cleanup has been included in the Report, I offer a few comments.

Cleanup Methods are Badly Confused by being mixed up with design (metal water-stops), consistency of concrete, vibration, keyway construction, job traffic, curing and "other operations."

Joint Cleanup is of utmost importance, especially in high concrete dams. It is regrettable that the Committee has placed its approval on air-water cutting of unset concrete; a method which has been used on most of the large concrete dams built in this country since 1933. I discussed this subject over a year ago (JOURNAL, June 1939, p. 188-3). My conclusions that this method was dangerous, should never have been specified or used and should be strictly prohibited, were based on observations made on a large concrete dam under construction by the U.S.B.R. An invitation was extended by the Institute requesting the U.S.B.R. to submit a discussion of my criticism of this method. Their spokesman R. F. Blanks, devoted 9 pages to this subject (JOURNAL, June 1939). It is notable that Mr. Blanks did not deny that the method was dangerous; he did indicate that is would be "strictly prohibited" on some future important Bureau work. His statement should be contrasted with the specifications (quoted below) covering air-water cutting of unset concrete on Grand Coulee Dam.

Contradiction. Near the top of page 348 the Committee states that if the joint must be water-tight, it may be worth while to embed a metal water-stop; however near the bottom of the same page this is entirely unnecessary since:

> . . . the joint may be adequately prepared for good water tight bonding by removal of the surface film and dirt by sand-blasting immediately prior to placing the new concrete.

Contradiction. Two cleanup conditions are covered:

> (a) Where laitance is present or the wetter consistencies of concrete have been used initial cleanup by air-water cutting of unset concrete is recommended.
>
> (b) No initial cleanup preparation is necessary on horizontal joints in mass concrete that has been placed at dry consistencies with the aid of vibrators. . . .

The next sentence states: ". . . it should be restored to the bright clean condition immediately following the initial jetting . . ." The Committee failed to note that there had been no "initial jetting" in this case.

Until the above contradictory statements are clarified, this portion of the Committee Report will remain inoperative. Air-water cutting of unset concrete should be eliminated from the Report or a warning inserted to the effect that it is a dangerous practice if used on concrete dams.

(*5*) *What is Laitance?*

The method of joint cleanup recommended by the Committee hinges on laitance on the concrete; this term is used 3 times (p. 348). The exact meaning of this term sometimes becomes of utmost importance as is illustrated below.

The first Grand Coulee Dam contract (U.S.B.R. Specification 570) provided for the cleanup of horizontal joints as follows:

> . . . all laitance and loose or defective surface concrete (shall be) removed by means of jets of air and water applied at high velocity. . . . The surface of each lift shall be washed immediately prior to the placing of the succeeding lift of concrete, to remove all foreign material which may have accumulated since the first cleanup operation: *Provided*, That where foreign material cannot be removed by washing, wet sandblasting will be required. . . .

Air-water "cutting" was done as early as four hours after the concrete was placed, using pressures considerably higher than the 100 p.s.i. limit set by the Committee; this removed ¼ to ½ in. of concrete. Before the next lift was placed, 3 or more days later, a surface discoloration appeared. The contractor was required to wet sand-blast nearly all joint surfaces in order to remove this discoloration; in many instances, due to delays in placing the next lift (beyond the usual 3-day schedule) wet sandblasting was repeated 2 or 3 times. Owing to a disagreement in the interpretation of "laitance" and "foreign material" the contractor is now endeavoring to collect from the Government (under this item alone) the sum of $770,211 for work not called for in the contract (Court of Claims No. 44659). This amounts to about 30¢ per sq. yd. of joint or 18¢ per cu. yd. of concrete.

Whether or not a coating which resulted from the action of natural agencies on concrete ingredients which had been furnished by the owner (cement), or tested and approved by the owner (aggregates and water) and as a result of the specified methods of "cutting" and curing (continuous sprinkling or spraying with water) was "foreign material" is a question for the courts to decide.

The second Grand Coulee contract (Specification 757, Nov., 1937) was worded as follows:

Method B. Upon completion of a concrete lift and before the fresh concrete has taken the final set the surface of the construction joint shall be cut with a high-velocity air-water jet to remove any defective mortar or concrete and to prepare the surface for proper bonding with the succeeding lift. . . . Immediately before placing the concrete of the succeeding lift, the entire surface of the construction joint shall be given a final cleanup, and laitance and defective concrete not previously removed, reset cuttings, coatings of calcium carbonate and other cement coatings, stains, debris, and other foreign material shall be removed by chipping and wet sandblasting. . . .

A sharp distinction has been made here between coatings and foreign materials.

Particular attention is called to this usage of "laitance." Any laitance present would remain unset until after "the fresh concrete has taken the final set," hence if the surface of the construction joint was "cut with a high-velocity air-water jet" it would necessarily be removed; therefore laitance as used here can apply only to something which was not present when the surface was cut "to remove any defective mortar or concrete."

The writer has collected hundreds of definitions and usages of "laitance" from the engineering literature of the past 100 years; it can be said with utmost assurance that there is *no precedent in the English language* for the usage of "laitance" as applying to something which forms on a block of concrete *after the top layer has been removed* by means of a "a high-velocity air-water jet."

Incidentally the methods of cutting and curing specified in both of the above contracts furnish a perfect setting for manufacturing vast quantities of calcium carbonate which was the principal constituent of the coating which the U.S.B.R. compelled the contractor to remove at his own expense.

The Committee should make clear whether "laitance" is being used with the meaning which has been established by the usage of the past 75 years or in the purely "private" manner promulgated by the U.S.B.R. in the specifications for Grand Coulee Dam which are now in force (March, 1940).

CLOSURE BY THE COMMITTEE

The points raised by Mr. Abrams in his discussion of the proposed report of Committee 614 do not differ greatly from those raised in his discussions of O. G. Patch's paper on mixer efficiency tests* and Charles E. Wuerpel's paper on durability of construction joints.† Since the points (a) to (f) in his section (2) on the mixer efficiency test and elsewhere on the joint cleanup are largely covered already in com-

*JOURNAL, Amer. Concrete Inst., June and September *Supplement* 1939; *Proceedings*, Vol. 35, p. 180-1 and 180-25.

†JOURNAL, Amer. Concrete Inst., June 1939; *Proceedings*, Vol. 35, p. 188-1.

ments on his earlier discussions in the same volume, a careful review of these first comments is suggested. However, the Committee believes a recitation of certain further facts may be desirable in concluding the discussion.

In connection with the degree of control in force at the Grand Coulee Dam, it should be repeated that there are three sizes of sand, and that specification limits for batching a typical 4-yard mix work out about as follows: water 1½ per cent or about 14 lb., cement 1½ per cent or 22 lb., sand 2 per cent or 75 lb., fine gravel 2 per cent or 54 lb., and other gravels 3 per cent or 62, 85, and 94 lb., respectively.

The sum of refinements afforded by the concrete-compression-test results in concrete control has made it possible to obtain a standard deviation in test results in this field work comparable with that in many laboratories. It should be kept in mind that, for the same effectiveness of control, as the number of tests is increased so does the chance of obtaining larger extremes of test results. Nevertheless, the coefficient of variation of 7.5 per cent obtained for the July 1937 tests mentioned compares favorably with coefficients of variation of 5 per cent to 7 per cent for test series made in various research laboratories, and with coefficients of variation as high as 12 per cent on smaller field work. On the basis of these recognized statistical standards, this is a good showing despite the fact that contributing factors were not the same throughout these periods. In addition to slight inaccuracies in batching resulting from the weighing tolerances indicated above, frequent changes were made in the proportions of coarse aggregate to make them conform with current plant production. Furthermore, and probably most important in causing variations in test cylinder strengths, was the practice of sampling frequently deliberately to avoid "average" batches. Accordingly, the extremes of consistency necessary in various parts of the work were represented in the extremes of the test results. Despite these factors, productive of variation in test results, there have been many consecutive runs of 10,000, 50,000, and 100,000 yards of concrete with a coefficient of variation of not over 5 per cent. Practical field men will recognize that on the average job, before one-tenth of these quantities has been produced, variation in test results will usually be well outside of these limits.

In connection with the mortar mix tests for mixer efficiency, two methods of obtaining mortar samples are described in the original paper by Mr. Patch* p. 175. The test, it may be noted, is not intended to measure the average cement, sand, and water content in the mix,

*JOURNAL, Amer. Concrete Inst., Jan. 1939; *Proceedings*, Vol. 35, p. 173.

which would require that a "representative" sample of the entire mix be taken, but is intended to reveal if and when the batch is so uniformly mixed that three grab samples, no matter where or how taken, would contain reasonably uniform sand-cement ratios and water-cement ratios.

The mortar-mix test for mixer efficiency has been consistently demonstrating the uniformity of mixing on many jobs for several years.

The fact that slightly higher water-cement ratios are generally found in the mortar samples than is provided for the batch as a whole is discussed at the bottom of page 177 of Mr. Patch's original paper. The main point is whether or not the tests of the three samples show uniformity, not whether they are precisely representative of the whole batch.

Use of 2.4 per cent and 4.5 per cent to express mixer errors would give a distorted picture of results. The uniformity of the tests is expressed by the values 102.4 per cent and 104.5 per cent. The figures selected by Mr. Abrams for criticism show the small spread of 0.021 among the three shifts or 2.1 per cent. Such small variations can at times be partially accounted for. For example, during periods when the gravel screening plant was working one or two shifts less than the mixing plant, the daily drawdown of stockpiles, bringing segregated materials to the mixers mostly during one shift, might easily account for somewhat greater variations on that shift.

Since publication of the report, others as well as Mr. Abrams have questioned the desirability of metal waterstops in construction joints. At the time the report was first written, the chairman was interested in cleanup of the construction joints in the thin arch Gene Wash and Copper Basin Dams in the Colorado River aqueduct system, and in these dams metal waterstop strips were used as an added precaution against leakage. As the usual difficulties in getting a good final cleanup were encountered, especially at the start, the added factor against leakage presumably afforded by the strips seemed an impressive protection. Nevertheless, according to the reports of engineers who have made extensive examinations of many concrete structures after long service, metal waterstop strips do not have a good record. They have either been proved unnecessary where good stiff unstratified concrete has been placed and the joint well cleaned, as indicated by the watertightness of the joints in modern dams so constructed, or they have been rendered useless by cracking around them or by seepage through the porous top lift of wet-placed stratified concrete. On the basis of this later information, waterstop strips have been omitted

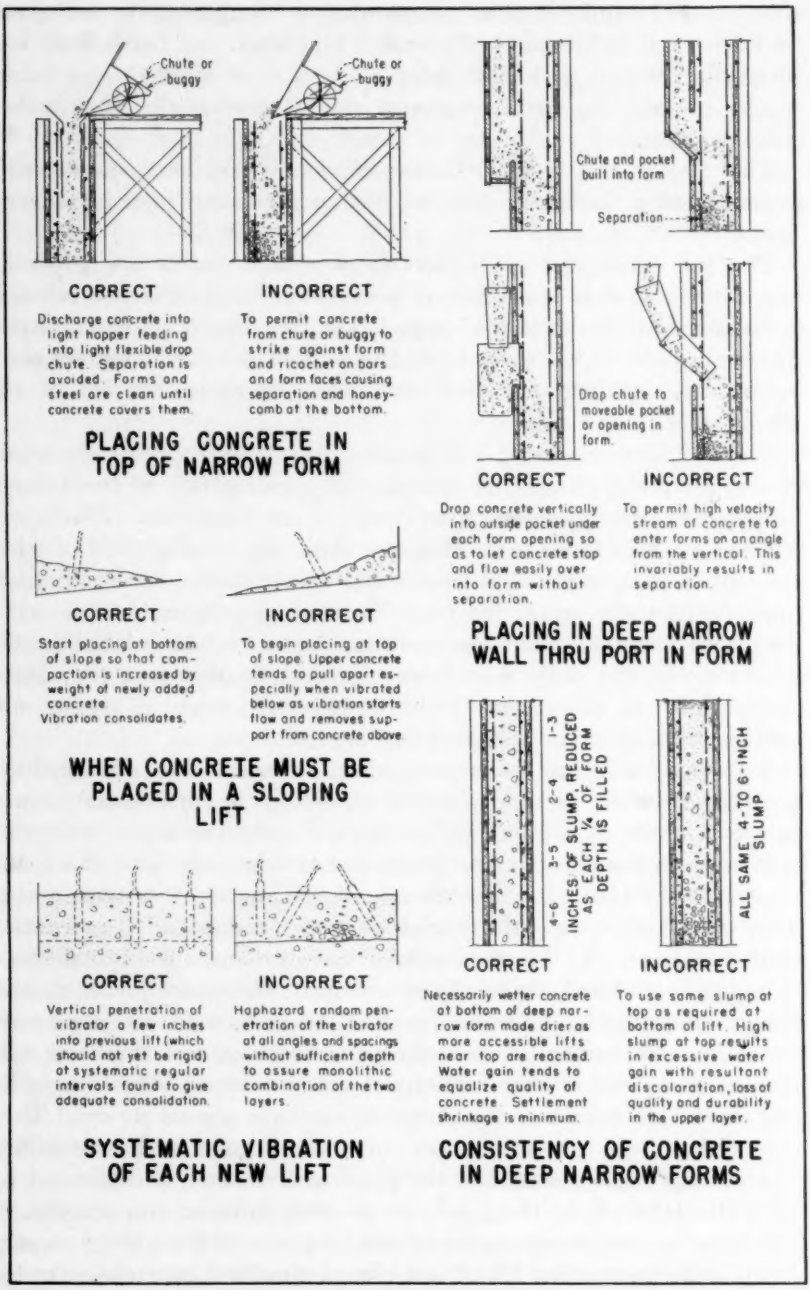
Chute or buggy
Chute or buggy
CORRECT
Discharge concrete into light hopper feeding into light flexible drop chute. Separation is avoided. Forms and steel are clean until concrete covers them.
INCORRECT
To permit concrete from chute or buggy to strike against form and ricochet on bars and form faces causing separation and honeycomb at the bottom.
PLACING CONCRETE IN TOP OF NARROW FORM
Chute and pocket built into form
Separation
Drop chute to moveable pocket or opening in form.
CORRECT
Drop concrete vertically into outside pocket under each form opening so as to let concrete stop and flow easily over into form without separation.
INCORRECT
To permit high velocity stream of concrete to enter forms on an angle from the vertical. This invariably results in separation.
PLACING IN DEEP NARROW WALL THRU PORT IN FORM
CORRECT
Start placing at bottom of slope so that compaction is increased by weight of newly added concrete. Vibration consolidates.
INCORRECT
To begin placing at top of slope. Upper concrete tends to pull apart especially when vibrated below as vibration starts flow and removes support from concrete above.
WHEN CONCRETE MUST BE PLACED IN A SLOPING LIFT
1-3
2-4
3-5
4-6
INCHES OF SLUMP, REDUCED AS EACH ¼ OF FORM DEPTH IS FILLED
ALL SAME 4-TO 6-INCH SLUMP
CORRECT
Vertical penetration of vibrator a few inches into previous lift (which should not yet be rigid) at systematic regular intervals found to give adequate consolidation.
INCORRECT
Haphazard random penetration of the vibrator at all angles and spacings without sufficient depth to assure monolithic combination of the two layers.
SYSTEMATIC VIBRATION OF EACH NEW LIFT
CORRECT
Necessarily wetter concrete at bottom of deep narrow form made drier as more accessible lifts near top are reached. Water gain tends to equalize quality of concrete. Settlement shrinkage is minimum.
INCORRECT
To use same slump at top as required at bottom of lift. High slump at top results in excessive water gain with resultant discoloration, loss of quality and durability in the upper layer.
CONSISTENCY OF CONCRETE IN DEEP NARROW FORMS

FIG. 3. CORRECT AND INCORRECT METHODS OF PLACING CONCRETE.

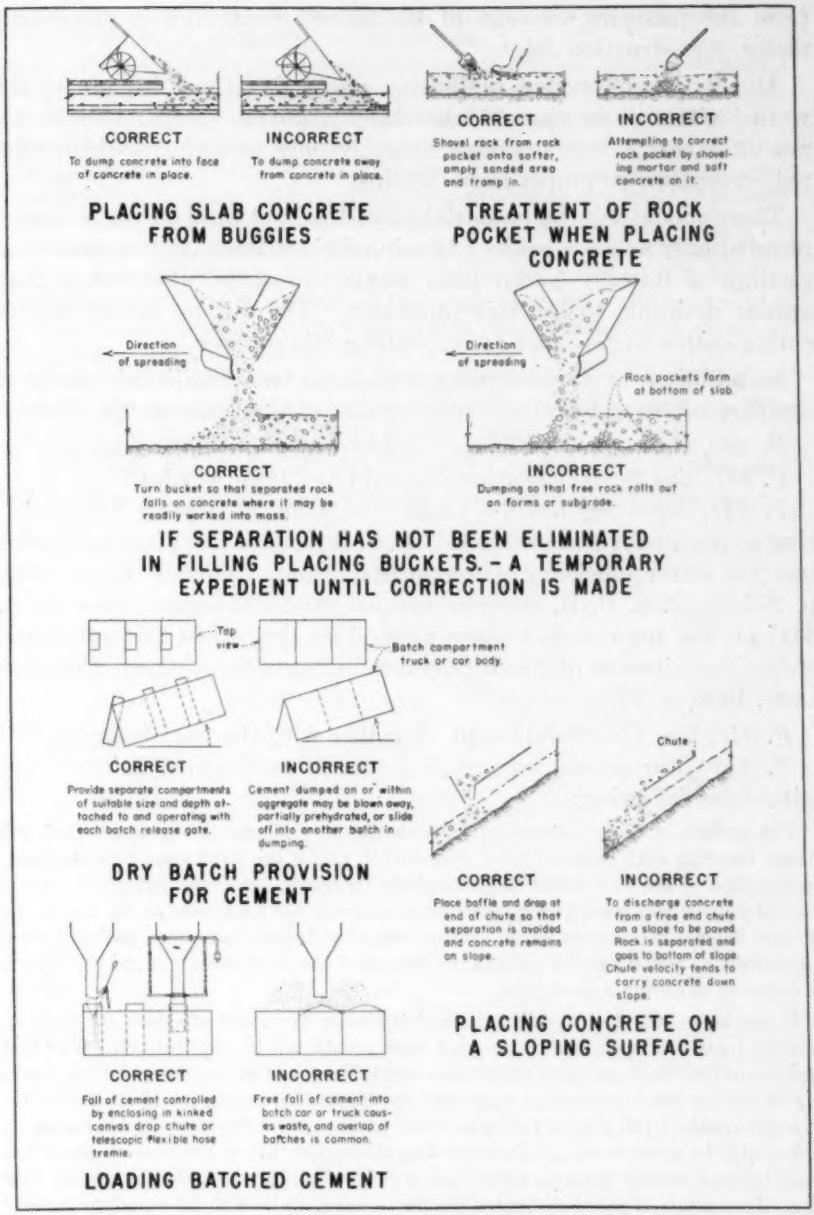
CORRECT
To dump concrete into face of concrete in place.
INCORRECT
To dump concrete away from concrete in place.
PLACING SLAB CONCRETE FROM BUGGIES
CORRECT
Shovel rock from rock pocket onto softer, empty sanded area and tramp in.
INCORRECT
Attempting to correct rock pocket by shoveling mortar and soft concrete on it.
TREATMENT OF ROCK POCKET WHEN PLACING CONCRETE
Direction of spreading
CORRECT
Turn bucket so that separated rock falls on concrete where it may be readily worked into mass.
Direction of spreading
Rock pockets form at bottom of slab.
INCORRECT
Dumping so that free rock rolls out on forms or subgrade.
IF SEPARATION HAS NOT BEEN ELIMINATED IN FILLING PLACING BUCKETS. - A TEMPORARY EXPEDIENT UNTIL CORRECTION IS MADE
Top view
Batch compartment truck or car body.
CORRECT
Provide separate compartments of suitable size and depth attached to and operating with each batch release gate.
INCORRECT
Cement dumped on or within aggregate may be blown away, partially prehydrated, or slide off into another batch in dumping.
DRY BATCH PROVISION FOR CEMENT
CORRECT
Fall of cement controlled by enclosing in kinked canvas drop chute or telescopic flexible hose tremie.
INCORRECT
Free fall of cement into batch car or truck causes waste, and overlap of batches is common.
LOADING BATCHED CEMENT
Chute
CORRECT
Place baffle and drop at end of chute so that separation is avoided and concrete remains on slope.
INCORRECT
To discharge concrete from a free end chute on a slope to be paved. Rock is separated and goes to bottom of slope. Chute velocity tends to carry concrete down slope.
PLACING CONCRETE ON A SLOPING SURFACE

FIG. 4. CORRECT AND INCORRECT METHODS IN HANDLING MATERIALS AND PLACING CONCRETE

from the following revision of the second paragraph in the report under "Construction Joints."

Also it is believed the ambiguity concerning the third use of the word "laitance" on page 348 has been removed. Apparently it was not clear that it was intended to apply only to surfaces which were not susceptible to proper initial jetting.

There should now be no mistake but that wet sand curing is recommended only as an expedient to minimize or eliminate laborious final cleanup of initially jetted joint surface cleanups wherever it may appear desirable to use this procedure. There is no better way of curing such a surface and accomplishing this purpose.

In addition to the following revisions, two additional plates of sketches of correct and incorrect methods are submitted, Fig. 3 and 4.

P. 330, lines 13-14—Change "a drawing is" to "drawings are."

P. 331, line 3—"the portions" should be "these portions."

P. 341, beginning line 7—Under "Measure of Mixing Efficiency" change remainder of sentence to: U. S. Bureau of Reclamation mortar test for mixer efficiency (*Proceedings*, Amer. Concrete Inst., 1939, p. 173, or U. S. B. R. concrete manual, first and second editions, p. 341) or, the Iowa State College method for the "Field Determination of the Constituents of Fresh Concrete" (*Proceedings*, Amer. Concrete Inst., 1930, p. 202).

P. 347, line 11—Should read "together with the use of aggregate."

P. 348—For second paragraph under "Construction Joints" substitute the following:

The surface of the construction joint should be prepared in a manner that will insure bonding with concrete later placed on it, and if the joint must be watertight, preparation of the joint must be particularly thorough. It is not difficult to obtain a good joint where good quality low-slump concrete has been used at the top of the lift and has not been overworked. Where wet consistencies have been used and often excessively vibrated, the concrete at the surface of the lift is usually so inferior that it is not easy to obtain a good joint.

In the latter case the so-called initial jet cleanup is a good expedient for such an inferior joint condition, which on good work would not be encountered. The first step should be the removal of all laitance and inferior surface concrete and the washing of mortar from protruding aggregate by means of a strong jet of air and water at approximately 100 pounds per square inch pressure after the concrete has hardened sufficiently to prevent the jet from raveling the surface below the desired depth and from forming cloudy pools of water that will leave a film on the surface when they dry. The surface of joints so treated should be especially well moist-cured, preferably by a 1-inch layer of saturated sand, and, if possible, the surface should never be permitted to dry out during the interval before concrete is placed upon it, at which time it should be restored to the bright clean condition existing immediately following the initial jetting by means of vigorous brushing with fine wire hand brushes or

by sandblasting as necessary. If the surface has been properly wet sand-cured, very little final scrubbing or sandblasting will be necessary.

Initial jetting, followed by a thorough final cleanup can also be used with good results where quality of concrete at the joint surface is good. It produces no better results, however, than can be obtained readily on surfaces which have been properly placed at the right consistency, by omitting any initial treatment and removing the surface film and dirt to a bright new surface by sandblasting immediately prior to placing the new concrete. The initial jetting method is usually found to be more expensive than the final sandblasting method and is not as foolproof. Final sandblasting without initial cleanup has been found to produce excellent results economically on horizontal joint surfaces of mass concrete that has been placed at dry consistencies with the aid of vibrators, if the surface concrete is protected until it has set from excess working due to keyway construction, endeavor to embed all coarse aggregate, and general job traffic.

In all cases the new concrete should be preceded by about one inch of soft mortar of the same proportions as that in the concrete. When accessible, this should be scrubbed into the surface of the joint with wire brooms.

P. 349, line 8—Under "Cold Weather Concreting", should read "dissipation of heat more rapidly than additional heat is generated".

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Ready-Mixed Concrete Operations in Philadelphia*

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MEMBERS AMERICAN CONCRETE INSTITUTE

SYNOPSIS

The paper first traces the history of ready-mixed concrete in Philadelphia and describes in some detail the economic considerations that have influenced its successful development. The nature of the market for this material is explained, followed by a discussion of various problems presented ready-mixed concrete producers by the user. A description of plants and organization is next given. Methods of technical control receive considerable attention in one of the leading chapters preceding the concluding paragraphs which outline several interesting studies that have been conducted in the interest of improvement of quality.

The ready-mixed concrete industry is a comparative youngster in the construction materials group, although it has out-grown its swaddling clothes. As a commercial industry it was born about 1921, but it did not become recognized as an integral and permanent branch of the building industry until about 1928 or 1929 and, consequently, its principal development has been during years of depressed business conditions. During the past ten years it has grown from a group of about 100 companies producing approximately 1,500,000 cubic yards to almost 700 companies, scattered through the United States and producing about 10,000,000 cubic yards of mixed ready-to-use concrete.

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The purpose of this paper is to say something about some of the problems which confront the ready-mixed concrete operator and the methods which have been used in their solution. We venture to discuss that subject since the company we represent, and with which we have been long connected, is a pioneer in the industry. Since our experience is solely with Philadelphia conditions we cannot presume to speak for the industry as a whole, but we believe that some of the things which we have learned will be of general interest and of general application. Further, it should be pointed out, our experience is almost wholly with central mixing operations, as opposed to transit mixing operations.

DEVELOPMENT OF INDUSTRY IN PHILADELPHIA

The first commercial ready-mixed concrete operation in Philadelphia was started by the senior author of the paper on March 17, 1921. That plant was one of the first in the country, if not the first, designed to serve the general market. It was a central mixing plant with a one cubic yard mixer. Cement was handled in bulk—shoveled by hand from box cars to the boot of a bucket elevator—and stored in a wooden-stave silo. For the batches cement was weighed on a platform scale and the aggregates were measured by volume. Concrete was transported in ordinary dump trucks. As crude as the operations were when viewed in the light of present-day practices, they represented an improvement over average current standards.

The incentive for the establishment of the plant was a major repaving program in Philadelphia. Traffic conditions in the city's narrow streets made the handling of bulk materials practically an impossibility and this new product, brought to the sub-grade ready to use, was a boon to the city as well as to the contractor. Streets laid unpaved a minimum of time and traffic was delayed much less than had job-mixed methods been used. Of course, that type of operation was not entirely unprecedented since there had been earlier experience with central mixing plants established by the Contractor for specific jobs.

From that small beginning about 19 years ago the industry has developed to the point where more than two-thirds of the concrete used in Philadelphia for all purposes comes from the ready-mixed concrete operators, equipped with machinery for accurate control of proportions and with modern transportation facilities. Two companies supply the market in Philadelphia. In the case of Warner Company, also a large producer of aggregates, the proportion of total tonnage marketed in Philadelphia represented by ready-mixed concrete sales has increased from about 50 per cent in 1930 to about 70

per cent during the past two or three years. It is believed that our experience in Philadelphia is not unlike that in other communities in which the ready-mixed concrete industry has developed—and that includes, with one notable exception, almost every metropolitan area in the country.

The development of the ready-mixed concrete industry in Philadelphia has been, in one sense, along somewhat restricted lines. For most of the important work the proportions for the concrete have been specified so definitely as to leave little lee-way for the operator to use such knowledge as he may have of design of mixtures. He is limited largely to the accurate filling of a prescription. While that condition has represented a curb on operators well equipped with technical personnel, it has on the whole been a good influence. The City Building Department, for example, has issued regulations governing the use of ready-mixed concrete which have gone a long way toward insuring the production of a product of high quality by the average producer. In many ways these regulations are unique and they are believed to be of sufficient interest to include as an *Appendix* to this paper. Among other things they limit ready-mixed concrete to that produced by the central mixing method—a policy with which the authors are in entire sympathy.

ECONOMIC CONSIDERATIONS AND MARKET

It cannot be said that when it was decided to start the operation of a central-mixed concrete plant in Philadelphia that a lengthy study of the economics of all phases of the operation was considered. As has been stated previously, the plant was erected to serve a street-paving program. It soon became evident that a successful commercial ready-mixed concrete plant must depend upon a broad market and serve all types of concrete operations.

What was the nature of the market in Philadelphia which could be served? The answer to that question was, and still is, that it is limited only by the economics of the particular situation—and the word economics must be used in the *broad, general* sense. If this new method of producing concrete was to survive, it must furnish this commodity more economically, in the *broad* sense, than the contractor or other user could produce it.

Many economic considerations, in addition to the tangible out-of-pocket expense items,* which came to mind have proven themselves over the years, not only in Philadelphia, but in the industry throughout

*Economics of Ready-Mix Versus Job-Mix Concrete, Report of Committee 610, R. L. Bertin, author-chairman, JOURNAL Amer. Concrete Institute, Jan.-Feb. 1936; *Proceedings* Vol. 32, p. 309

the country. Some of them are not always acknowledged by the user. They are:

1. The degree of control over the quality of the concrete. The established ready-mixed concrete operator, of any standing, is equipped with accurate weigh batchers, water measuring devices, facilities for heating materials in cold weather, etc., which are likely to be absent in the case of job-mixed concrete except on the larger jobs. The ready-mixed concrete operator can, therefore, furnish the small and moderate sized job with facilities for control which are economically available only on big projects.

2. The opportunity for technical control. The ready-mixed concrete operation can provide for a quality of inspection not ordinarily feasible on the job, except in the case of the larger projects—and this is particularly true of the central mixing plant.

3. The stand-by service. The ready-mixed concrete operator has the capacity to handle peak yardages which in the case of job-mixed concrete would require excessive investment in plant.

4. Saving of space. The saving of space in congested areas is an important consideration.

5. The convenience value; an intangible difficult to evaluate but nevertheless important. The contractor at once delegates an important part of his responsibilities for estimating, supervision, etc. to the shoulders of another.

There are many other so-called "intangibles" which might be mentioned but these should serve as illustration. Obviously these items, as well as the items of out-of-pocket costs, affect different jobs differently. In the last analysis, ready-mixed concrete must show an advantage to the user or he will mix his own concrete.

It is evident that the advantages of ready-mixed concrete are more apparent in the case of the smaller job than for the larger one. Not only is there economy in its use, but a vastly higher quality of product will result under average conditions. For the small job, say 500 cubic yards or less, plant investment and operation is relatively high, even though the plant is simple. The money saving lies, for the most part, in that fact. The convenience value is given due recognition. The advantages on the side of quality are equally obvious. The small job is not likely to be equipped with accurate proportioning equipment; frequently there are no specifications governing the quality of the concrete; and if there are, little attention is paid to them; and there is practically never any inspection. On the other hand, the

well-equipped ready-mixed concrete operator has all of the facilities for control available to the larger job. As a result architects and responsible contractors have come to recognize that ready-mixed concrete, from a reliable plant, is far more dependable than the uncertain output of the average small mixer. An outstanding result has been the marked improvement in quality of concrete going into small construction work.

On the large job, justifying a modern plant, the importance of the intangibles diminishes and the relationship of out-of-pocket costs predominates. The experienced contractor is capable of producing a high quality concrete and he is capable of gearing his plant costs to his needs much more nearly than can be done in the small job. The selection of product, therefore, usually depends on a realistic analysis of the costs involved. Nevertheless, central mixed concrete has superseded job-mixed concrete almost entirely on important structures erected in Philadelphia during the past several years.

On much work, bridge construction for example, attended by delays for excavation, stripping of forms, and steel erection resulting in lost time for equipment and high job-handling costs, the ready-mixed concrete operator can and does present a desirable source for concrete to the contractor. The industry finds its keenest competition on road work in suburban areas. Space and traffic congestion are not important factors in many cases. Railroad siding set-ups, if within reasonable distance of the center of the job, offer economical batching facilities because of the low cost of transporting materials and erecting storage and proportioning equipment. This competition has been successfully met in Philadelphia because of the ability of the ready-mixed concrete operation to handle large yardages on short notice and to keep a steady flow of concrete to the men on the job.

While the "intangibles" diminish in importance in the case of the large job, they are by no means absent in many cases. In citing some of them which we believe deserve consideration, we point out again that our principal experience has been with centrally mixed concrete.

1. Long association of the cement, water, and aggregates produces added cohesiveness; the mass becomes more thoroughly lubricated. Cement finishers have remarked that central-mixed concrete possesses a very desirable cohesive quality which aids finishing.

2. Consistent uniformity is obtained by central-mix methods at a concrete plant having adequate facilities for proper storage, grading, and classification of aggregates; accurate weigh-batching; regulation of temperature in cold weather; control of water-cement ratio; etc.

3. Greater plasticity is obtained by central-mix methods due to the violent action in the initial mixer, which discharges the pre-mixed concrete into the specially designed truck drum. The following mild agitation in the revolving drum en route to the job more completely hydrates the cement and "densifies" the concrete into a homogeneous mass. The benefit to the contractor is in the easier and more economical placement of the concrete and easier finishing, which saves time and money, and in the avoidance of "stand-by" lost time both of labor and equipment.

4. Larger pours are possible through the use of central-mix concrete where a plant of sufficient capacity is available. This is particularly true in congested districts, where job facilities are limited, and applies on both small and large jobs. Furthermore, a contractor can expand his business and work on several jobs simultaneously, without increased capital investment in mixers and equipment.

5. Reduced labor costs are effected by the contractor when he uses central-mix concrete. He can start a pour at any hour of the day, within reason, and due to the greater yardage obtained per hour can complete the pour with little or no over-time. In wintertime this is particularly advantageous, as he eliminates the necessity of heating materials on the job and can arrange his pour during the warmest part of the day.

So far as area of market is concerned, in Philadelphia it has been found that the weighted average haul (one way) over the years has ranged from 3.5 to 5.5 miles. Since 1932, when but two plants have been in operation by one of the Philadelphia producers, the average haul ranged from 4 to 4.5 miles one way until 1938 and 1939, when by reason of developing a new outlet for concrete in Philadelphia (namely, concrete foundations for small house construction) the average haul rose to 5.5 miles. Prior to 1932 four plants made possible the distribution of concrete to the whole Philadelphia area on a 3.5 mile average one-way haul. It is not to be supposed from this record that longer hauls are not feasible or are not made. In a metropolitan area such as Philadelphia, however, with railroad sidings scattered throughout the entire area and with business normal, the hauls are bound to be comparatively short to insure maximum service with minimum capital investment in transporting units.

To sum up, there is a real economic value to the community by reason of a well established ready-mixed concrete industry. The centralization of mixing operations has greatly simplified the problem of inspection by the numerous supervisory authorities. The valuable

experience gained by years of specialization on a vitally important construction material has become more readily available to engineers, architects, and all users of concrete. The ability of the ready-mixed concrete operator to deliver the concrete simultaneously directly to points most convenient for final placement has increased its utility in building work and reduced the time required to complete construction projects. These circumstances produce conditions for the successful promotion of enduring concrete.

PROBLEMS PRESENTED INDUSTRY BY USER

If the ready-mixed concrete operator is to serve the wide market available to him he must be prepared to furnish a wide variety of mixtures. He is much like the druggist—a prescription filler. Variations must be met in gradings and proportion of fine to coarse aggregates, in cement and water content, and often these differ for the same grade and use of concrete. The "out-of-town" engineer, who does not care to use the local grading of aggregates or the rules of the local bodies having jurisdiction over construction, tends to complicate the ready-mixed concrete operator's problem. Unfortunately, with an increase of federal projects this problem is becoming more and more acute.

During 1939, Philadelphia central mixing plants furnished more than 64 distinct classes of concrete and, what is more significant, the variety of mixes called for on any normally busy day represents a fairly high percentage of the total. A brief summary, indicating the sources from which all these mixes emanate follows:

Bureau of Building Inspection	4 classes
Bureau of Highways	3 classes
Pennsylvania State Highway Dept.	3 classes
Philadelphia Transit Dept.	3 classes
Bureau of Engineering	4 classes
Board of Education	6 classes
U. S. Navy Yard	10 classes
U. S. Treasury, Procurement Division and Phila. Housing Authority	6 classes
Miscellaneous top coats, Pennsylvania & Reading Railroads, individual architects' specifications, more than	25 classes
	64 classes

These numerous specifications relieve the ready-mixed operator of most of the responsibility for the basic design of mixes, but not of his duty to interpret specifications honestly and intelligently to the end that the quantities employed always conform to the intent and pur-

pose of the author of the design. Long experience has demonstrated that very few specifications are free of omissions, inaccurate premises, conflicting requirements and clauses which can be construed in more than one way. Any success which has attended the development of central-mix concrete in Philadelphia may be credited to the competent discharge of this duty, plus, of course, an efficient follow-up of technical control and inspection over the physical operations.

Engineers and inspectors, representing supervising authorities, are frequently assigned to enforce specifications, thereby ostensibly relieving the concrete producer of the responsibilities referred to. Nevertheless we have found the problems only tend to increase in complexity because of the unfamiliarity with characteristics of our local aggregates and too little experience with or knowledge of many of the principles governing the science of concrete. If patience, good faith and a sound, working knowledge of his subject are possessed by the producer, however, the proper solution of every problem posed by vague or contradictory specifications is always attainable. To fill every prescription accurately and efficiently is the task which a ready-mixed concrete operator must accomplish and the expenditure of conscientious effort to meet that test is truly an exhilarating experience.

PLANTS AND ORGANIZATION

Philadelphia is bounded on the east by the Delaware River. Through the heart of the city passes the Schuylkill River. The principal supplies of sand and gravel used in Philadelphia come from the vicinity of Morrisville, Pa., and from the Delaware River. They are transported to the Philadelphia market by water. Hence all ready-mixed concrete plants in Philadelphia are located on the Delaware or Schuylkill rivers. Five plants, 3 on the Delaware and 2 on the Schuylkill rivers, operated by the two companies, supply the demands of Philadelphia and its suburbs.

Fig. 1 shows the Berks Street concrete plant of Warner Co. Here sand and gravel are unloaded from barges, placed in ground storage and, as needed, transported by belt conveyor to overhead bins. Cement comes in by railroad in bulk and is unloaded by screw conveyors and a Fuller Kinyon pumping system to a concrete silo or to several day tanks located overhead from which it is transported by screw conveyors to the weighing batchers.

Our company has four central mixing plants all of which were constructed 10 years or more ago and consequently do not contain many of the modern appliances, such as the electric eye, which are

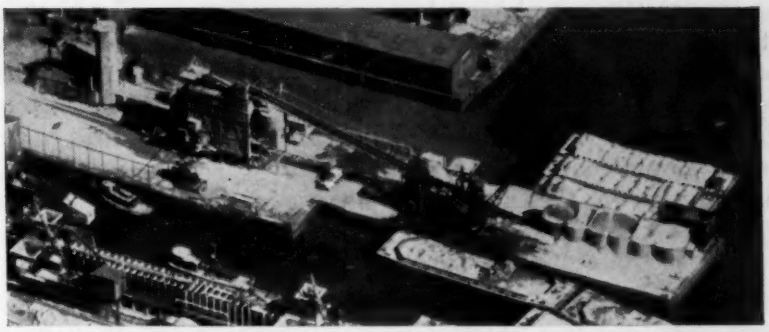

FIG. 1—BERKS STREET PLANT OF WARNER COMPANY, PHILADELPHIA

to be found in recent installations. Nevertheless, the mixers, weighing machinery, conveyors, batchers, etc., have been well maintained so that the plants continue to operate efficiently. Our mixing equipment consists of three 2-cu. yd., two 3-cu. yd. and one 4-cu. yd. units, capable of producing about 500 cu. yd. of concrete per hour. During the peak seasons a volume of from 4500 to 5000 cu. yd. frequently has been furnished on busy days.

All our plants are, of course, equipped to furnish heated concrete during cold weather. The method employed is to heat the water before it enters the mixer, although steam is available to the aggregate bins to heat the sand and gravel when necessitated by extremely low temperatures. General practice is to maintain a temperature of about 70° in the concrete whenever atmospheric temperature is below 40°.

Our transportation units range from standard gas engine driven agitating bodies to special bodies driven from the main truck engine. Their capacities as agitators are, in the main from 4½ to 6 cu. yd. Concrete is delivered in either the standard agitator revolving at about 4 r.p.m. or the specially designed bodies, with blades or vanes of low profile, revolving at about 2 r.p.m. The slowly revolving body (2 r.p.m.) prevents segregation, grinding of aggregates and an undue reduction in slump between the plant and the job.

Our delivery market is divided into districts, each of which is under the direction of a superintendent whose duty it is to see that all construction projects in his division obtain satisfactory service at an economical cost to the company. Such superintendents are required to have practical experience in the operation of a concrete plant and in construction. As soon as a new job enters the excavation stage or clearing of the site begins, the district superintendent confers with

the construction superintendent to plan an efficient and economical set-up for handling the ready-mixed concrete. This involves a safe roadway for our heavily loaded trucks and a suitable dumping box, usually large enough to contain two full loads of concrete. Always, if at all possible, arrangements are made for a one-way road so that the trucks can continue from dump to exit without causing delay to inbound vehicles. Much thought and effort is given to this planning as a quick turn-around for trucks at both delivery and loading points is the essence of economical trucking and fast service.

Throughout the successive stages of each project, the district street superintendent maintains close contact with all activities, specifies the number of trucks required to serve each pour, and keeps our central office closely informed regarding progress, as well as requirements for the immediate future. This enables our delivery headquarters to make definite plans for truck assignments in various sections of the city so that our fleet of 75 concrete trucks is properly distributed among the several mixing plants to take care of demands without lost motion or confusion. On large pours, one of the street superintendents stays on the job more or less constantly to speed up the dump and get-away, and he keeps in constant contact with the central delivery office. Of utmost importance is the check of the yardage required toward the end of the delivery so that the exact number of batches is dispatched from the plant. The street superintendents function under the general direction of the traffic and service manager at operating headquarters.

The truck department, under the jurisdiction of a superintendent of trucks, with his staff of assistants, foremen, and inspectors is responsible for the mechanical operation of the truck fleet, the hiring and discipline of the drivers, repairs, painting, etc. However the work assignments for all trucks are made by the traffic manager, independently of that department.

The whole service structure is presided over by the traffic manager. Assisting him at service headquarters is a staff of order clerks working at a modern telephone order turret served by four trunk lines and direct wires to all mixing plants. The order room is equipped with a teletype system, with stations at the various mixing plants, over which all orders are dispatched.

The order department is the nerve center of the distributing organization. Here all orders are received, interpreted, classified, dispatched, and followed up, and here are received inquiries, complaints, and reports from street superintendents and others. All truck assign-

ments are cleared through this department. The whole intricate network involved in the process of delivering hundreds of batches of concrete of different proportions to their proper destinations on time is kept under this centralized control.

Each mixing plant has its own dispatching or shipping office. Orders for specific trucks are relayed to the different mixer positions and, after loading, delivery tickets are furnished the drivers. A daily truck record is kept, showing essential information covering the day's deliveries. Statistics of individual truck costs and delivery costs on each project and numerous miscellaneous items of information such as waiting time, etc., are maintained in a delivery record and cost division. That record is alive and constantly used and furnishes us with the information to keep a check upon the efficiency of the component parts of our distribution organization.

Of course, to permit the organization just described to function, the order for the concrete must be obtained. Our company constantly canvasses the architects and engineers with a technical man or with one having an intimate knowledge of building materials. Specifications are distributed which are suitable for incorporation in the job specification and which furnish details of grading of materials and proportions of concrete in keeping with the applicable code, thus assuring the owner protection in the matter of quality.

When specifications for a job are available they are checked to determine their compliance with the applicable regulations and their requirements for grading of aggregate, type of cement, and design of mix, including water-cement ratio. Cases are found where the specifications far exceed the requirements of the various codes. In some instances aggregates are specified which are unusual in the Philadelphia area or are unfitted for the use to which they are to be put. This information is scanned and if the requirements are not standard or are unfitted for the work, steps are taken, through the engineer, to correct them. Before a quotation is made all elements of cost are given most careful consideration. Embodied in the quotation is a statement of the conditions under which the concrete will be furnished —i. e., proportions, maximum slump, quantity guaranteed per hour, and other factors which assure a mutual understanding.

TECHNICAL CONTROL

All factors touching upon the quality of the concrete produced by the two companies in Philadelphia is under the direct supervision of independent consulting and testing engineers, retained by the companies. In the case of the Warner Company, one of the authors of

this paper (Mr. Knopel), not an employee of the company, together with a staff of six assistants, has complete charge of control of proportions and water content of the concrete. He and his organization make routine tests of all of the materials going into the concrete as well as regular tests of the strength of the concrete. He has full authority to reject materials which he considers unsuitable and to increase amount of cement used above the designed mix when special circumstances require such action to maintain the W/C ratio. In addition, he conducts numerous special investigations as a guide to maintaining and improving quality and economy.

A field laboratory is maintained at each plant. It is equipped with all of the necessary apparatus for making tests essential to immediate control. Equipment is available for sieve analyses, moisture and specific gravity determinations, determinations of silt, colorimetric tests for organic impurities, molding compression cylinders, making slump tests of the concrete and, in the winter, temperature tests of the concrete. At the principal plant there is a Los Angeles rattler, a standard flow table, a laboratory mixer, a thermostatically controlled oven and the usual complement of scales, balances, graduates, burettes, etc. Fig. 2 is a view of the Berks Street field laboratory.

Compression tests of concrete, routine cement tests, chemical tests and other special determinations are made, using the facilities of an established commercial testing laboratory.

One of Mr. Knopel's staff is stationed at each active mixer at all times. In Fig. 3 one of the mixer positions is illustrated with the inspector visible at his station. All of these men have had technical training and, as required by the regulations appended hereto, have "given to the Bureau of Building Inspection satisfactory evidence of his experience and qualifications to make such certification [as required by the regulations] over the individual signature of the inspector certifying."

These inspectors supervise the weighing of the cement and aggregates and personally measure the mixing water, the quantity of which they themselves have determined. In other words, they have a personal responsibility and do more than routine work. They make regular determinations of moisture, aggregate grading and consistency to guide them in fixing the water content of the individual batches.

The control of consistency in our operations is facilitated by the fact that, almost always, two or three mixer batches comprise the load for the agitator. The experienced operator, and the inspectors

FIG. 2—BERKS STREET FIELD LABORATORY

FIG. 3—MIXER OPERATOR AND INSPECTOR

referred to *are* experienced, can judge the consistency of the first batch with a high degree of accuracy and make necessary corrections in the second. The agitation en route will incorporate the several batches into a homogeneous mass of the desired slump. The aggregates delivered to the retail yards, as a rule, conform to rather close grading tolerances. The change in water to correct the second batch is, therefore, very slight—at most amounting to not more than a gallon per cubic yard.

In the control of proportions the engineering inspectors have, of course, more than their personal knowledge on which to depend. Results of past experiences and of the special investigations referred to later, have been correlated in form for ready reference. For example, the quantity of water required for the many classes of concrete has been accurately charted together with the amount and nature of the correction which needs to be made to meet various conditions as they arise. Intimate knowledge of the characteristics of the wide range in cements and aggregate gradings which must be used by any large ready-mixed concrete operator is essential to the maintenance of uniformity of quality. For example, different cements have different water demands and require different minimum mixing times. The same is true of different gradings of aggregates.

Maintenance of adequate records is a most important adjunct to control of quality. All test data are available to any one with a proper interest. A daily log of each batch of concrete is kept by the independent testing organization showing the size of batch, the size of the aggregate, the kind of concrete, the slump, the total water content, the number of the truck, the time of dispatch and the purchaser.

Mr. Knopel's technical engineering service is always available as liason between the purchaser and the producer. He aids in reviewing specifications and arriving at mutually satisfactory interpretations of ambiguous requirements.

MISCELLANEOUS INFORMATION AND SPECIAL INVESTIGATIONS

Information gained from the routine tests carried out in the control of our product and from the special investigations conducted from time to time have been of outstanding assistance to us in producing concrete of high and uniform quality in an economical manner. It seems worth while to touch, very briefly, on such studies as illustrating one of the many ramifications of a controlled ready-mixed concrete operation.

1. Temperature Loss and Recession in Slump

Table 1 gives results of an investigation of temperature loss in transit made ten years ago, while in *Table 2* there is shown a very recent study of both temperature losses and recession in slump occurring between the plant and the job.

It may be desirable to explain how we deal with the additional one gallon of water per cubic yard required to be added at the plant due to the recession in slump of 1 in. during transit between the plant and destination. This one gallon excess may raise the W/C ratio 0.2 gallons per sack of cement. Under existing regulations the only recognized W/C ratio is that obtained at the plant mixer and all our mixes are necessarily so fixed as to the cement content that these regulations are fully satisfied.

2. Yield Tests

Since 1933 we have made numerous checks of the yield of concrete. We have employed for that purpose a seamless steel tube measuring $9\frac{25}{32}''$ inside diameter, $72\frac{1}{16}''$ long, large enough to contain a half sack batch of concrete and we have since checked many batches of a larger number of mixes for yield. The absolute volume method of computation for yield has been proved substantially accurate, but, due to the tendency to bleeding or creation of free water in wet mixes which obviously does not enter into yield of hardened concrete, we do not count water in excess of 4.4 cu. ft. per yard as yield, or, stating it more positively, in the range of mixes with cement factors below $6\frac{1}{2}$ sacks we have set a minimum of 22.6 cu. ft. by absolute volume of solids as required to yield a cubic yard of fresh concrete.

3. Effect of Plant Mixing Time on Composition of Concrete after ½ Hour of Agitation in Agitator Body

The tests were conducted on concrete mixed at the plant in five different periods, viz.: 15 seconds, 30 seconds, 45 seconds, 60 seconds and 90 seconds, following which, in every case, the concrete was transported in an agitator body revolving at 4 r.p.m. for a half hour and delivered to the same destination. Three samples were obtained at the plant mixer, representative of the discharge at the beginning, middle and end of the batch and three additional samples of the identical concrete from the agitator body upon discharge at destination, obtained in the same sequence. All samples were analyzed for composition of the four constituents by the Dunagan method and results were recorded in terms of variation from the designed mix.

TABLE 1

E. L Conwell & Co., Philadelphia, Pa.

Dec. 4, 1929

Field Test No. 1030

Air	Temperature (°F) Concrete (Plant)	Time of Haul (Min.)	Concrete (Destination)	Concrete (In Place)	Loss at Destination	Total Loss Immediately After Placing
°F	°F		°F	°F	°F	°F
30	88	40	85	82	3	6
29	65	30	63	63	2	2
29	65	35	62	61	3	4
15	77	10	76	76	1	1
17	77	15	77	76	1	1
17	71	25	70	68	1	3
18	68	45	58	58	10	10
15	60	35	57	57	3	3
18	88	35	83	81	2	7
19	85	20	82	80	3	5
20	78	10	77	77	1	1
25	73	25	70	68	3	5
27	90	30	88	87	2	3
26	85	30	84	84	1	1
27	88	30	87	87	1	1
28	100	25	97	87	3	3
28	106	20	103	103	3	3

TABLE 2—TEMPERATURE LOSSES AND RECESSION IN SLUMP DURING TRANSIT—CENTRAL MIX CONCRETE

Load No.	Air Temp. (°F)	Temp. at Plant (°F)	Slump at Plant (inches)	Time in Transit (min.)	Temp. on Arrival (°F)	Slump on Arrival (inches)	Time Unloading (min.)	Temp. at End of Unloading (°F)	Slump at End of Unloading (inches)
1	20	68	2¾	8	68	2¾	18	68	2½
2	17	65	6½	31	63	5½	72	60	4½
3	20	69	6½	49	68	6¼	14	67	6¼
4	22	62	6¼	44	60	5½	55	58	4½
5	25	62	4	23	61	2¾	55	62	2
6	18	70	4½	37	68	3½	50	69	3
7	21	70	4	8	68	3¼	28	68	3
8	25	60	4	20	58	3	40	58	3
9	24	70	4½	30	68	3½	60	67	3
10	24	60	5½	37	56	4½	..	56	4½
11	22	70	6½	25	64	6	145	60	2
12	26	59	6½	9	60	6¼	56	58	5¼
13	26	71	6½	22	69	6½	38	68	5½
14	26	75	4½	10	70	1¾	33	70	1½
15	26	70	5	8	68	3¾	21	68	3½
16	18	70	4½	30	68	3½	60	67	3
17	22	60	5½	25	56	4½	..	56	4½

These results apparently indicated that if plant mixed concrete is subsequently agitated for ½ hour in an agitator body at about 4 r.p.m., composition of the mass will then closely approach the designed mix irrespective of the period of the initial mixing given at the plant.

4. Sand Studies

We wanted to determine the ideal grading of our sand which would produce the greatest workability when used as the fine aggregate in concrete. The tests were conducted on both mortar and concrete mixes and included studies of slump, flow, density, W/C

ratio, bleeding action and strength. A typical grading of our sand is given below in the left hand column and an ideal grading according to the tests in the right hand column:

	% Passing	
	Typical	Ideal
⅜ in. Sieve	100	100
No. 4 "	95	95
8 "	79	82
16 "	65	58
30 "	45	36
50 "	14	19
100 "	2	2

The significant difference between the ideal grading and the typical grading is the more regular distribution of the various sizes. It will be observed that the predominance of the 30 to 50 sizes has been reduced in the ideal grading.

5. Lime Admixture Studies

We have made several series of tests to determine the effect of substituting hydrated magnesium lime or lime putty for a portion of the cement in various mixes. These tests, which have been repeated at intervals during the last four years, all clearly indicate that a marked improvement occurs in mixes, leaner than 6 sacks when such substitution is made up to 7½ per cent of the cement by weight, equivalent to 15 per cent by volume. Plasticity and cohesiveness are very notably improved; also there is an increase in slump on a comparative basis, e. g., 470 lb. cement and 37½ gal. total water produces a slightly stiffer consistency than 445 lb. cement, 25 lb. lime and 37½ gal. total water, aggregates the same quantities for both mixes.

Strength of the concrete has never shown a decrease either at 28 days or 90 days, as a result of lime substitution within the limits mentioned, and usually indicates an increase. Our earlier studies antedate recent research on this subject in the laboratory of the Massachusetts Institute of Technology.

In concluding this paper the authors desire to express their appreciation for the opportunity of presenting an account of the status of ready-mixed concrete in the construction industry in Philadelphia to members of the Institute. They cannot omit a grateful acknowledgment to Stanton Walker, Engineering Director of the National Sand & Gravel Association, for his counsel and assistance in its preparation.

APPENDIX

REGULATIONS GOVERNING USE OF CONCRETE MADE AT CENTRAL MIXING PLANTS

Definition: Concrete which is mixed at such distances from point of depositing as to necessitate the use of approved vehicles other than buggies, chutes, or similar contrivances, is considered as central plant mixed concrete.

As the result of field and laboratory tests, directed by and under the supervision of the Bureau of Building Inspection, the use of concrete made at central mixing plants is approved for construction work in Philadelphia, subject to the following conditions and requirements:

1. Ready-mixed concrete shall be made in plants located within the City of Philadelphia. Representatives of the Bureau of Building Inspection shall at all times have unrestricted access to the plants, materials, records and all other data pertaining to mixing, transferring, use and inspection of ready-mixed concrete.

2. All materials entering into the mix and the concrete mixed at a central plant shall be certified by thorough and competent independent inspection approved by the Bureau of Building Inspection.

3. The central plant shall be equipped to properly and accurately measure the materials entering into the concrete mix. Owners shall present to the Bureau of Building Inspection evidence of their competency to carry out in full the requirements of this approval.

4. The cement shall be measured by weight and when purchased in sacks shall be taken from original 94-pound bags as supplied by the manufacturer. No bag of cement shall be divided for mixing. Aggregates shall be measured by weight. Ingredients shall be measured by weight when in the dry state, and by volume in calibrated containers when in the liquid state. Under no conditions shall materials be measured by "shovel-count." The plant shall be equipped with all necessary apparatus for verifying scales and measuring devices. Concrete shall be mixed in an approved rotary mixer equipped with a timing device which will automatically lock the discharge lever during the mixing period and release it at the end of the full time of mixing.

5. The concrete shall be mixed one and one-half minutes as a minimum, with the drum rotating an approved number of revolutions. Larger batches of concrete may be required to have a longer time for mixing.

6. Cement and aggregates shall meet the current A. S. T. M. specifications. Each shipment of cement shall be tested. Complete tests of the aggregates shall be made daily. The water shall be clean, free from oil, vegetable substances, etc., and the tensile strength of 1-3 mortar gauged with this water shall be not less than that developed with distilled water when mixed in the same proportion using Ottawa sand. Tests for moisture content, clay, silt, and organic matter and sieve analysis of the aggregates shall be made at intervals sufficient to control the quality of products and proportions and consistency. Aggregates must be tested for moisture content within three hours. Tests of aggregates for organic matter shall be made from deliveries before they are unloaded in the plant.

7. Every truck load of aggregate or concrete mixed in a central mixing plant to be used in a structure erected under the jurisdiction of the Bureau of Building Inspection shall be certified as having the required and approved proportions, and

that it is thorough mixed. This certification shall be made on a written report made by an independent inspector engineer stationed at the plant, who has given to the Bureau of Building Inspection satisfactory evidence of his experience and qualifications to make such certification over the individual signature of the inspector certifying. The responsibility for the filing of this information each day central mixed concrete is mixed and transferred rests with the plant, and failure to so certify it automatically rejects the product as acceptable for use in structures.

8. The central mixed concrete shall be discharged into clean, water-tight bodies, and shall be hauled in rotary or agitator trucks approved by this Bureau.

9. Concrete from central mixed plants shall be deposited at the site where it is intended for use at a period not to exceed sixty (60) minutes from the time it leaves the central plant mixer and deposited in forms within thirty (30) minutes. The time of initial set of the cement used in the mix shall always be greater by certified test than the elapsed time between mixing of the concrete and the depositing in the forms.

10. A delivery ticket shall accompany each load of ready-mixed concrete, setting forth the class or type of concrete being delivered, and the exact time the concrete was discharged from the central plant mixer into the delivery truck. This ticket shall be signed by the inspector at the plant. Upon reaching the job, each certified load of concrete shall have the exact time of dumping noted on the ticket and a check mark placed opposite the class of concrete delivered. The resident inspector on the job shall certify that the class and quality of concrete delivered conform to the laws and Regulations of the Bureau of Building Inspection. These tickets shall be filed with the plant, and at all times be accessible for examination by the Bureau of Building Inspection.

11. In freezing weather, or when temperatures below 40 degrees F. prevail concrete shall be delivered and deposited in the forms at temperatures not less than fifty (50) degrees F., nor more than one hundred (100) degrees F.

12. When in the judgment of the Bureau of Building Inspection load tests are required they shall be of such number as is adjudged necessary. They shall be made in accordance with the laws and regulations governing such tests and test procedure in the City of Philadelphia.

13. Wherever in the judgment of the Bureau of Building Inspection it is for the best interests of construction not to use plant-mixed concrete, this approval does not apply.

14. All laboratory tests shall be made by laboratories of recognized standing, and acceptable to the Bureau of Building Inspection.

15. Failure to comply with the provisions of the terms of this approval will be considered as withdrawal of approval without further notice.

Approved: February 7, 1930.

(Signed) LEMUEL B. SCHOFIELD, Director
Dept. of Public Safety

Discussion to close in June 1940* JOURNAL *should reach A. C. I. Secretary in triplicate by Apr. 1, 1940.

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Discussion of a paper by Foster, Knopel and Whitten:

Ready-Mixed Concrete Operations in Philadelphia*

BY C. A. MENZEL, HERBERT J. WHITTEN, L. H. TUTHILL, HARRY C. SHIELDS, STANTON WALKER, ALEXANDER FOSTER, JR.

CONVENTION DISCUSSION

Carl A. Menzel†—How do your inspectors at the central mixing plant know the concrete will be at the consistency ordered when it reaches the delivery point?

Mr. Whitten—A partial answer to that question will be found in the tables in the paper, showing studies made of the extent of recession of slump based on various lengths of haul. Briefly the answer is that over the years we have determined the extent of that recession, and knowing what recession to expect under the various conditions—such as temperature, and length of haul, we are able to give sufficient additional slump at the plant to insure that on arrival at destination, the consistency will be such as is desired.

Mr. Menzel—Does that always work for different cements?

Mr. Whitten—In the last several years, in handling modern cement we have learned that different cements do have differences in their water demand. Inspectors at our mixers are acquainted with the brand and type of cement being used. Adjustments do have to be made to take care of the variations of cement. That is a matter of knowledge, particularly through experience in making deliveries and partly through experience in trying constantly to obtain by our little pilot batches, that we run in our laboratories, the necessary information.

*JOURNAL, Amer. Concrete Inst., Feb. 1940.
†Portland Cement Association, Chicago.

Mr. Menzel—How great is the recession? What do you do if you miss the desired slump by more than the consumer or the engineer wants?

Mr. Whitten—It would be risky for any one to claim infallibility in this business, but the tables accompanying the paper show that for 17 deliveries taken at random, we obtained a certain consistency at the plant, the inspector went along with the truck and measured the consistency when it arrived at destination, at the time of arrival, and then, following the complete discharge of the batch of concrete, again obtained the consistency of the last of the discharge. Our variation was approximately an inch. An inch variation is well within the range of what is practical in slump measurement. Occasionally some one asks us to deliver concrete whose slump is specified in terms of a quarter or half an inch. Most of those who have had long experience with concrete and slump, think in terms of an average slump such as four to six inches. But if the slump varies from that desired at the job, there would be nothing to do but return the batch and not use it. It is up to us as marketers of concrete to deliver the consistency at the destination which the nature of the work requires. That is a responsibility we cannot escape, just as the responsibility is ours to furnish a mix of the right proportions.

Mr. Menzel—How long would you say that the concrete would retain its consistency between mixing and final placing? I believe the average haul is between three and five miles; what does that mean in terms of time in the city of Philadelphia?

Mr. Whitten—There has been a pretty common meeting of minds in the acceptance of an hour and a half as the safe interval between the contact of the water with cement at the mixer and the final delivery of concrete at the operation. Sometimes this is interpreted as including placement; in other cases is not. In actual practice it is very rare that we have to make delivery at such a distance that we cannot reach destination and discharge the concrete within about one hour.

*L. H. Tuthill**—How does added increment of water figure in the water-cement ratio? Do you include it in the water-cement ratio?

Mr. Whitten—I refer to the statement in the paper on that point (p. 367). It is a very important one. That is a somewhat controversial point and I'd better read the paragraph verbatim:

> It may be desirable to explain how we deal with the additional one gallon of water per cubic yard required to be added at the plant due to the recession in slump of 1 in. during transit between the plant and destination. This one gallon excess may raise the W/C ratio 0.2 gallons per sack of cement. Under existing regulations the only recognized W/C ratio is that obtained at the plant mixer and all our mixes are necessarily so fixed as to the cement content that these regulations are fully satisfied.

*Bureau of Reclamation, Denver.

In amplification, there are some differences of opinion among many of us as to whether or not the addition of water so required actually should be charged against the water cement ratio. However, we are operating in the city of Philadelphia under regulations of various public authorities and the only water-cement ratio which any of those authorities recognizes is the water-cement ratio at the mixer; consequently it is incumbent upon us to use a cement factor which will produce a water-cement ratio required by either regulations or interpretation of specifications. We have to take that penalty in added cement.

Mr. Tuthill—I have read that paragraph, but the last sentence was not clear, and that is why I asked the question.

*Harry C. Shields**—Frequently I have encountered controversial questions arising with reference to the total quantity of concrete supplied. Notwithstanding the signed delivery ticket obtained from the recipient of the concrete. Very frequently consideration has not been given on the part of the purchaser to deflection in form work and variation in grade. Has Warner Company encountered such a controversy, and if so what has been found to be the most effective means of satisfying the user that he has received the quantity of concrete which he is supposed to have received or you are supposed to have furnished?

Mr. Whitten—I think every producer of ready-mixed concrete is constantly faced with that problem. Our compnay, over the last 18 or 19 years, has had frequent controversies regarding the quantity of concrete supplied. I am glad to say that there appears to be less and less of that sort of argument. Immediately upon receipt of such a complaint, we send to the site of the work an experienced construction field man who is on our staff, to check the measurement of the concrete in place. It is quite surprising how very often we are able to settle those controversies by the simple expedient of taking exact measurements and convincing the complainant that he is in error. Some years ago we had what I think was the most serious complaint regarding the quantity of concrete furnished on a fairly large job where we had rather expected such a complaint, because we knew of careless practices in placing the concrete. The operation was so large and so diversified that it would have been almost impossible to measure the entire job, involving about 20,000 cubic yards of concrete. So we suggested to the purchaser that a certain portion of the work be cored, because we wanted to convince him that the measurements shown on the drawings for the job had not been followed in placing the

*Marquette Cement Manufacturing Co., Chicago.

concrete. The purchaser agreed to that, and a coring machine was placed on the job under a friendly agreement that whichever side was wrong would be obligated to pay for the cost of coring the work. So we cored the section of the work that was selected and insofar as we were concerned, were preparing to continue the coring operation, not only on that section of the job but other sections. But the results of the cores, measured after some three or four days work, showed so conclusively that the thickness was so much in excess of the designed thickness, that the complainant contractor asked the coring operations be suspended, and not only assured us that he was completely satisfied regarding the quantity of the concrete that had been delivered to him, but actually apologized to us for having raised the question in the first place. On occasions when users not entirely accustomed to ready-mixed concrete have discussed with us the question of yield, we like to have the contractor or his representative come to our field laboratory where we have fairly adequate facilities for running trial batches of the mix and demonstrating that the quantities employed in the mixer which are to be used on the operation actually do produce the quantities represented. By such methods there is frequently established a more satisfactory bond of confidence between the maker of the concrete and the user.

*Stanton Walker**—I should like to ask Mr. Whitten if he can offer any average figure which would represent the difference between the theoretical volume of concrete, as figured from the plans, and the actual volume of concrete required to fill the forms? I think that question is presented to me as frequently as any other. I think we have all had experience which indicates that forms do have a tendency to spread or to be slightly inaccurate or a slab a quarter of an inch too thick. In your experience how much difference is there between the theoretical and actual volume? How much over-run must be allowed for?

Mr. Whitten—That is a large order that Mr. Walker has assigned. There are two general classes of construction served with ready-mixed concrete: building projects of all sorts and kinds and paving and road work. My personal experience with road contractors has been that they seem to expect an over-run varying from three to five per cent. I cannot give a very specific answer as to building work. Some contractors want possibly up to three per cent over-run, but against that I am puzzled a little by the surprising fact that many contractors, especially on very large projects, have reported to us that the concrete ordered and received on the job is almost identical with the amount of

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concrete specified in the plans. Part of that, I think, can be explained by construction involving reinforcement, which of course is not compensated for in computing the quantity of concrete. I am sorry I cannot give a more specific answer.

Mr. Foster—I have been responsible to a great extent for the units involved. In a structural cubic yard there are two things we try to check on—a cubic yard of 27.54 cubic feet of fluid concrete, and as shown in the paper, the fact that we try to have between 22.5 and 22.6 cubic feet of solids in a cubic yard. We hear very little complaint. If there is a complaint, we find some reason for it as Mr. Whitten has indicated, in measurements or that some of the concrete has been thrown away. On road work we add three per cent—27.81 cubic feet to the cubic yard. On a job about thirty miles from Philadelphia where we erected a plant and operated a truck mixing unit, there were two contracts comprising bridge work and pavement. When the contractors had finished their bridge work and started on the road work, they insisted on a cubic yard of 27 cu. ft. They said "That is what we are being paid for and we do not want this enlarged yard." So we entered into a contract with them stipulating that a cubic yard was to be composed of so many pounds of cement, cubic feet of sand and coarse aggregate and so many gallons of water, and they agreed with us that that amount of material produced a cubic yard of plastic concrete consisting of twenty-seven cubic feet. When the job was completed, one contractor had ordered more concrete for the run of the road than the other one. We observed the grade very carefully. One ran over one and a half per cent, and the other, less careful with his sub-grade, two and a half per cent.

Mr. Walker—I want to ask a question; I think I know the answer, but I want to be sure that everybody else understands it. Is ready-mixed concrete sold on the basis of an actual cubic yard of concrete or on the yardage of the concrete in place—as measured in place?

Mr. Whitten—A cubic yard of concrete delivered. In marketing concrete we represent that we deliver one cubic yard consisting of twenty-seven cubic feet of fresh concrete delivered to the site of the work, and we do not assume responsibility that the concrete, after being placed in such forms as the contractor provides, will necessarily agree with the plans for that form work, because we have no control over that.

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The Design of Concrete Mixes*

By Charles T. Kennedy†

SYNOPSIS

This paper presents a rational method for the design of the concrete mix with respect to workability. A "workability factor" is developed which has a similar relation to the workability of the concrete as the water-cement ratio has to the strength. The author finds that for any given cement and water-cement ratio, this factor depends upon the relative quantities of cement, water and aggregates and upon certain easily determined physical characteristics of the aggregates.

Instead of the original data upon which the method is based, its validity is demonstrated by application to an independent series of tests.

Examples are given of the application of the method to problems of design and to the selection of the proper aggregate, and attention is called to certain implications with respect to economy and quality.

A RATIONAL BASIS FOR THE DESIGN OF CONCRETE MIXES

The history of the rational design of concrete begins with the publication, in 1918, of "Design of Concrete Mixes," by Duff A. Abrams. This presented two new concepts: (a) the water-cement ratio law, and (b) an arbitrary function of the grading of the aggregate known as the "fineness modulus."

Experience has demonstrated the essential soundness of the water-cement ratio law. Not only the strength, but other principal characteristics of the concrete, have been found to be primarily dependent upon the ratio of the amount of mixing water to the amount of cement. The fineness modulus has not fared so well. It has been found to be demonstrably unsound, and has generally been abandoned.

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The water-cement law is not, in itself, a complete method of design. Abrams expressed the law in the form $S = \frac{A}{B^x}$, where S is the compressive strength of the concrete and x the water-cement ratio expressed in terms of cubic feet of water per sack of cement. He was careful to point out a qualification which has been emphasized by later investigators, namely: this law holds only as long as the mix is workable. A and B, in the formula, are constants whose values,—under standard conditions of mixing, curing and placing,—depend only upon the quality of the particular cement in use and the age of the specimens; they are independent of any other characteristics of the mix, such as the amount or grading of the aggregate. It is assumed, of course, that the aggregate is physically sound and the concrete free from injurious amounts of extraneous matter

It is obvious that the choice of the proper water-cement ratio is only the first step in the design of the mix. For a complete method of design, some means must be provided to determine the workability of the mix with regard to the amounts and grading of the aggregates. It was for this purpose that the fineness modulus was developed. Since the abandonment of this method, nothing has been advanced to take its place; even the Portland Cement Association has nothing better to offer than the mixing of trial batches,—the method of "cut and try."

Herein is presented a rational approach to the problem of the workability of the mix,—a method of design developed for use under actual working conditions and so used in the writer's practice over a period of years.

The fundamental law of the workability of the mix is this: for a mix to be workable, the volume of cement paste,—i. e., the absolute volume of the cement plus the volume of the mixing water,—must be at least equal to the volume of the voids in the dry rodded mixed aggregate.

Concrete in which the volume of cement paste was no more than equal to the volume of voids would be workable only under a strictly laboratory definition; it would have a slump theoretically equal to zero. For practical purposes, an excess of cement paste is required.

The second law of the workability of the mix is this: for any required degree of workability, the necessary excess of cement paste depends (a) upon the consistency of the cement paste itself,—lower water-cement ratios requiring larger excess amounts than higher ratios,—and (b) upon the surface area of the aggregate,—the larger the surface area, the greater the excess required.

The logical basis of these laws is illustrated, diagrammatically, in Fig. 1. Let diagram A represent adjacent particles of the dry rodded mixed aggregate, the black areas representing the voids. The action of the rodding is to compact the material, forcing the particles into such a condition of internal equilibrium that the sum total of the voids is reduced practically to the minimum consistent with the grading. When cement and water are added, the action of mixing the concrete has practically the same effect upon the aggregate. In a published series of tests (to be referred to in detail later), yield was reported as the ratio of the volume of freshly mixed concrete to the

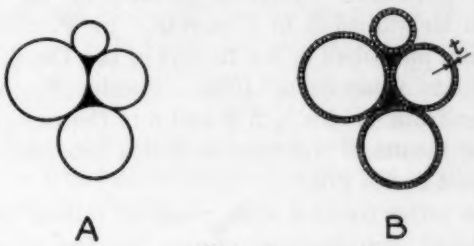

FIG. 1

apparent volume of the dry rodded mixed aggregate. In 37 cases in which the volume of cement paste was less than the volume of the voids in the aggregate,—the ratio varying from 0.75 to 0.99,—the average yield was reported as over 98 per cent. Allowing for some loss in evaporation and handling, this indicates that the apparent volume of the aggregate is substantially the same in the dry rodded condition and in the mixed concrete. Consequently, diagram A may also be taken to represent a section of freshly mixed concrete in which the amount of cement paste is just sufficient to fill the voids in the dry rodded aggregate,—the black areas now denoting cement paste. For this amount of paste, and for any smaller amount, the particles of the aggregate will evidently be in the same intimate contact as in the dry rodded condition, the internal friction will be high, and the concrete will be harsh and unworkable.

If an additional amount of cement paste is added to the concrete and uniformly distributed throughout the mass, the effect will be as indicated in diagram B; the individual particles of the aggregate will be forced apart, each particle being surrounded by a film of cement paste. For small thicknesses of this film, t, with respect to the particle diameter, the black areas in diagram B will be seen to be substantially equal to those in diagram A (i. e., to the volume of voids in the dry rodded mixed aggregate), while the cross-hatched areas

which represent the cement paste film are closely proportional to the surface area of the aggregate. It is the action of the cement paste film in reducing the internal friction that imparts plasticity to the mix, and this effect is obviously dependent upon the consistency of the paste itself. Parenthetically, it may be noted here that the average thickness of the cement film is of the order of magnitude of one one-thousandth of an inch.

The application of these laws will be demonstrated by means of an absolutely independent series of tests. All experimental data used in this article will be taken from one of the finest experimental investigations ever published—"Further Studies of the Water-Cement Ratio Strength Relationship in Concrete," by F. R. McMillan and Wm. R. Johnson, published in the Report of the Director of Research, Portland Cement Association, 1928. Specifically, the data used herein are taken from Tables 1, 2, 3 and 6 of the report, summarizing respectively the results of the tests in Series 186, Series 212 Group 1 and Group 2, and Series 213.

The first two series cover a wide range of mixes. Series 186 contains 228 different combinations of one grading of sand with one grading of gravel and a similar grading of limestone. The resulting mixes vary from 1-1-0 to 1-3-7 and 1-4-3, with water-cement ratios varying from 0.49 to 1.16. Series 212 Group 1 consists of 220 different mixes of two gradings of sand (size 0-14 and size 0-4) with two gradings of gravel (size 4-1½ and size ¾-1½) and with the same two gradings of limestone; the water-cement ratios vary from 0.49 to 2.53. Series 212 Group 2 contains 94 different mixes, duplicating certain of the mixes of Series 186 with a sand of slightly different grading; both gravel and limestone are used as coarse aggregates, and the water-cement ratios vary from 0.59 to 1.12.

Data given for each mix include: nominal mix (sacks of cement to cubic feet of dry rodded fine and coarse aggregates measured separately), true mix (sacks of cement to cubic feet of dry rodded mixed aggregate), unit weight of aggregate (pounds per cubic foot of dry rodded mixed aggregate), water-cement ratio, slump and flow of concrete, and 28-day strength. Of fundamental importance in the investigation is the observer's opinion of the workability reported for each mix; the following excerpt from the Report describes the methods used in this connection:

Consistency and Workability: During the molding of the specimens, careful observations were made by an experienced observer as to the workability or non-workability of the mixes. All the mixes are identified either by the letter "W" or "N" depending upon the classification in which they fall. The observer classified the mix according to his judgment of its behaviour in the mixing pan, on the flow

table, and in the mass as the slump cone was removed, as well as during the placing in the cylinder mold. Attention was also given to the size of the batch, the record indicating whether, in the observer's opinion, the concrete would be workable in job-size batches. Coincident with these observations, consistency of the mix was determined by both the standard cone and the slump table.

To calculate the volumes of the cement paste and voids, two values not given for these series were assumed. The absolute volume of a sack of cement was taken as 0.49 c. f. The specific weight of the aggregate was taken as 165 lb. per cu. ft., corresponding to the observed value of the specific gravity of 2.65 given elsewhere in the report for similar materials. With these assumptions, the volume of cement paste per sack of cement becomes x plus 0.49, where x equals the water-cement ratio in cubic feet per sack. The volume of voids in the aggregate becomes $\frac{(165 - w)N}{165}$, where w equals the unit weight of the dry rodded mixed aggregate in pounds per cubic foot, and N equals the true mix in terms of cubic feet of dry rodded mixed aggregate per sack of cement.

On this basis, calculations of the volumes,—cement paste and voids in aggregates,—were made for each of the 542 mixes included in the three series of tests. The volume of cement paste failed to exceed the volume of voids in the aggregate in 111 mixes. In accordance with the first law of workability given above, all of these mixes should be classified as "non-workable;" reference to the report shows that they have been so classified in 108 of the 111 cases. The three cases in which the calculated result varied from the reported opinion are summarized in the following table:

Series	Ref.	True Mix	Unit Wt.	Voids	x = w/c	Vol. Cement Paste	Slump
212-1	36	1-7	126	1.65	1.08	1.57	0.2"
212-2	169x	1-5.3	121	1.41	0.85	1.34	0 7"
212-2	197	1-7	125	1.70	1.05	1.54	0.5"

An inspection of these data, with particular reference to the low slumps reported, will make it obvious that these are indeed borderline cases, and can in no sense be regarded as upsetting the balance of evidence presented by the agreement in the other 108 cases.

The closeness with which this criterion may be applied is illustrated by the tabulated excerpts which follow.

The first three sets of tests above illustrate the effect of increasing the amount of water while maintaining constant proportions of cement and of fine and coarse aggregates. In the fourth set, a nominal

Series	Ref.	Mix		Unit Wt.	Voids	x = w/c	Vol. C. P.	Slump	W or N
		Nom.	True						
186	105	1-2½-5	1-6.4	129	1.39	0.77	1.26	0.1	N
	109	"	"	"	"	0.84	1.33	0.3	N
	114	"	"	"	"	0.91	1.40	0.2	N
	158	"	"	"	"	0.97	1.46	1.6	W
186	130	1-3-4	1-6.0	129	1.29	0.79	1.28	0.1	N
	133	"	"	"	"	0.85	1.34	0.2	N
	137	"	"	"	"	0.92	1.41	0.5	W
	169	"	"	"	"	0.99	1.48	1.2	W
212-2	181	1-2½-4	1-5.7	125	1.39	0.80	1.29	0.3	N
	183	"	"	"	"	0.93	1.42	3.5	W
	185	"	"	"	"	1.05	1.54	5.5	W
186	135	1-3-0	1-3.0	113	0.95	1.00	1.49	8.4	W
	136	1-3-3	1-5.2	128	1.16	0.95	1.44	1.3	W
	137	1-3-4	1-6.0	129	1.30	0.92	1.41	0.5	W
	138	1-3-5	1-6.9	131	1.38	0.89	1.38	0.2	N
	139	1-3-6	1-7.7	129	1.67	0.88	1.37	0.1	N
212-2	18x	....	1-7	115	2.12	2.18	2.67	1.3	W
	19x	....	"	128	1.57	1.58	2.07	1.1	W
	20x	....	"	126	1.65	1.16	1.65	0.2	N

water-cement ratio has been maintained, only the proportion of coarse aggregate being varied. In the last set, the same true mix has been maintained with varying proportions of fine to coarse aggregate, and with amounts of water adjusted to maintain the same nominal flow. In all five sets, a close relation is evident between workability and the relative amount of cement paste.

The data given in the various tables of series 186 and 212 cannot be used directly for the quantitative determination of the effect of the excess cement paste upon the workability of the mix. Due to the effect of the consistency of the cement paste, data based upon a constant water-cement ratio are necessary. Because of the wide range of water-content, the use of gap gradings, and the high incidence of harsh and unworkable mixes, Series 212 Group 1 is entirely too diffuse for the purpose. While the data for Series 186 and Series 212 Group 2 are arranged in sets of a uniform nominal water content, the actual water-cement ratios for each set vary considerably. All aggregates were used in a room-dry condition; since the same amount of mixing water was used for each mix in a given set, the correction for the absorption of the varying amounts of aggregates results in a varying water-cement ratio. A group of tests having a nominal water content of, say, 7 gallons per sack will present values of w/c from 0.77 to 0.89.

It is possible, however, to obtain the desired information by a process of interpolation. In general, each mix appears in each series with several different values of the water-cement ratio. By plotting slump against this ratio, a series of curves is obtained, one for each mix, from which the slump may be estimated for any required water

content. A few typical curves, obtained in this manner, are given in Fig. 2; attention is called to the typical S shape of the reversed curve, which always appears when sufficient points are available for plotting.

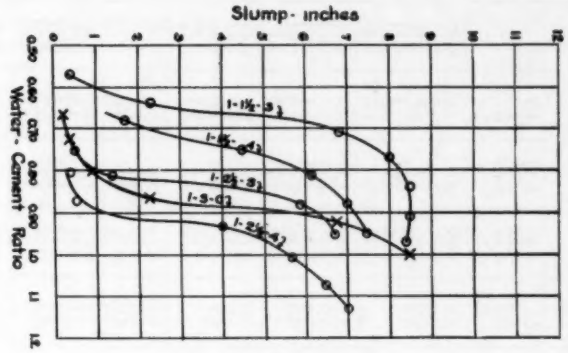

FIG. 2

A similar process of interpolation has been carried out by the authors of the report, and the results given therein in their Fig. 8, 9, 15 and 16. Separate diagrams are given for each of four water-cement ratios,—namely 5, 6, 7 and 8 gals. per sack,—for each of the two series of tests and for each of the two types of coarse aggregates. The values of the slump given in the accompanying Table 1 have been taken directly from these diagrams of the report.

The following notes, referring to the column headings of Table 1, explain its construction in detail.

Nominal Mix—Sacks of cement, cubic feet of dry rodded fine aggregate, cubic feet of dry rodded coarse aggregate; taken directly from the report.

True Mix—(*N*)—Cubic feet of dry rodded mixed aggregate per sack of cement. This value is given directly in the report, but only to two significant figures; thus, the nominal 1-1-2 mix is given as a 1-2.6 true mix. To obtain a greater degree of refinement in the calculations, the true mix has been computed. The unit weight of the coarse aggregates is not given in the report; given the unit weight of the fine aggregate, the true mix, and the unit weight of the mixed aggregate, it is possible to compute the unit weight of the coarse aggregate. By taking the average of this result for each of the mixes given in the Report, the unit weights were determined as 108 lb. per cu. ft. for the gravel in both Series 186 and 212, 105.9 lb. per cu. ft. for the limestone in Series 186, and 104.4 lb. per cu. ft. for the limestone in Series 212. The true mix was then computed on the basis of these unit weights. The values of the true mix given in Table 1 are these computed results, which in general agree to two significant figures with those given in the report.

Unit Weight—(*w*)—Weight of dry rodded mixed aggregate in lbs. per cu. ft.; taken directly from the report.

TABLE 1

	Nominal Mix	True Mix	Unit Weight	Voids	Surface Area	5 gal. per sack $V = 1.16$			6 gal. per sack $V = 1.29$			7 gal. per sack $V = 1.42$			8 gal. per sack $V = 1.56$		
		N	w	v	S	$V-v$	K	Slump	$V-v$	K	Slump	$V-v$	K	Slump	$V-v$	K	Slump
Series 186—Gravel	1-1-0	1.00	113	0.32	1790	0.84	4.7	10.4	0.97	5.4	10.9						
	1-1-1	1.73	128	0.39	1880	0.77	4.1	10.0	0.90	4.8	11.0						
	1-1-2	2.55	129	0.56	1980	0.60	3.05	9.3	0.73	3.7	9.7						
	1-1-3	3.47	126	0.82	2080	0.34	1.65	7.1	0.47	2.25	8.0						
	1-1½-0	1.50	113	0.47	2690	0.69	2.55	9.3	0.82	3.05	10.8	0.95	3.55	11.2			
	1-1½-2	2.99	129	0.65	2880	0.51	1.75	7.6	0.64	2.2	9.4	0.77	2.65	9.9			
	1-1½-3	3.82	129	0.83	2980	0.33	1.10	5.1	0.46	1.55	8.3	0.59	2.00	9.0			
	1-1½-4	4.77	126	1.13	3070	0.03	0.10	1.2X	0.16	0.50	6.0X	0.29	0.95	7.4			
	1-2-0	2.00	113	0.63	3590	0.53	1.50	5.8	0.66	1.85	9.9	0.79	2.20	11.1	0.93	2.60	11.2
	1-2-1	2.67	125	0.65	3690	0.51	1.4	3.0	0.64	1.75	9.2	0.77	2.1	10.6	0.91	2.45	10.9
	1-2-2	3.45	128	0.77	3780	0.39	1.05	1.3	0.52	1.4	8.0	0.65	1.7	9.6	0.79	2.1	9.9
	1-2-3	4.30	128	0.97	3880	0.19	0.5	0.5	0.32	0.8	6.3	0.45	1.15	8.2	0.59	1.4	8.4
	1-2-4	5.10	129	1.11	3970	0.05	0.15	0.1	0.18	0.45	4.0	0.31	0.8	6.8	0.45	1.1	7.5
	1-2-5	5.98	128	1.34	4060							0.08	0.2	4.2X	0.22	0.55	5.8
	1-2½-0	2.50	113	0.79	4490	0.37	0.8	1.0	0.50	1.1	5.8	0.63	1.4	9.4	0.77	1.7	10.6
	1-2½-2	3.86	129	0.84	4680	0.32	0.7	0.3	0.45	0.95	4.0	0.58	1.25	7.7	0.72	1.55	9.0
	1-2½-3	4.60	132	0.92	4780				0.37	0.75	1.8	0.50	1.05	6.4	0.64	1.35	7.6
	1-2½-4	5.45	131	1.12	4880				0.17	0.35	0.5	0.30	0.6	4.0	0.44	0.9	6.3
	1-2½-5	6.37	129	1.39	4960							0.03	0.05	0.5	0.17	0.35	3.6
	1-3-0	3.00	113	0.95	5370	0.21	0.4	0.1	0.34	0.65	0.8	0.47	0.9	6.8	0.61	1.15	9.0
	1-3-2	4.37	127	1.01	5570				0.28	0.5	0.5	0.41	0.7	4.2	0.55	1.0	7.5
	1-3-3	5.18	128	1.16	5670				0.13	0.25	0.1	0.26	0.45	1.0	0.40	0.7	6.0
	1-3-4	5.97	129	1.30	5760							0.12	0.2	0.5	0.26	0.45	3.0
	1-3-5	6.70	131	1.38	5840							0.04	0.05	0.3	0.18	0.3	1.7
Series 186—Limestone	1-1½-2	2.98	128	0.67	2880	0.49	1.7	5.9	0.62	2.15	8.6	0.75	2.6	9.2			
	1-1½-3	3.80	128	0.85	2980	0.31	1.05	1.8	0.44	1.5	6.6	0.57	1.9	7.3			
	1-1½-4	4.78	124	1.19	3070				0.10	0.3	3.0X	0.23	0.75	5.0			
	1-2-1	2.65	125	0.64	3690	0.52	1.4	2.0	0.65	1.75	8.5	0.78	2.1	10.1	0.92	2.5	10.5
	1-2-2	3.42	128	0.77	3780	0.39	1.05	0.8	0.52	1.4	6.8	0.65	1.7	8.8	0.79	2.1	9.3
	1-2-3	4.18	130	0.89	3880	0.27	0.7	0.4	0.40	1.05	4.0	0.53	1.35	6.7	0.67	1.75	7.3
	1-2-4	5.07	128	1.13	3970				0.16	0.4	0.3	0.29	0.75	3.7	0.43	1.1	6.4
	1-2-5	6.04	125	1.46	4060										0.10	0.25	1.9
	1-2½-2	3.89	127	0.90	4680				0.39	0.85	1.4	0.52	1.1	6.0	0.66	1.4	7.6
	1-2½-3	4.65	129	1.01	4780				0.28	0.6	0.3	0.41	0.85	3.5	0.55	1.15	6.3
	1-2½-4	5.47	129	1.19	4880							0.23	0.45	1.4	0.37	0.75	4.8
	1-2½-5	6.35	128	1.42	4960							0.0	0.0	0.3	0.14	0.3	0.9
	1-3-2	4.31	128	0.97	5570				0.32	0.55	0.3	0.45	0.8	1.6	0.59	1.05	6,7
	1-3-3	5.13	128	1.15	5670				0.14	0.25	0.1	0.27	0.5	0.7	0.41	0.7	4.5
	1-3-4	5.96	128	1.34	5760							0.08	0.15	0.0	0.22	0.4	1.3
	1-3-5	6.78	128	1.52	5840										0.04	0.05	0.5

TABLE 1—Continued

	Nominal Mix	True Mix	Unit Weight	Voids	Surface Area	5 gal. per sack $V = 1.16$			6 gal. per sack $V = 1.29$			7 gal. per sack $V = 1.42$			8 gal. per sack $V = 1.56$		
		N	w	v	S	$V-v$	K	Slump	$V-v$	K	Slump	$V-v$	K	Slump	$V-v$	K	Slump
Series 212.2—Gravel	1-1½-0	1.50	114	0.47	2710	0.69	2.55	9.7	0.82	3.0	11.0	0.95	3.5	11.0	1.09	4.05	11.0
	1-1½-2	3.02	128	0.68	2900	0.48	1.65	8.0	0.61	2.1	9.5	0.74	2.55	10.2	0.88	3.05	10.3
	1-1½-3	3.96	125	0.96	3000	0.20	0.65	4.6X	0.33	1.1	7.9	0.46	1.55	8.7	0.60	2.0	9.0
	1-1½-4	4.86	124	1.21	3100				0.08	0.25	5.8X	0.21	0.7	7.2	0.35	1.1	7.0
	1-2-0	2.00	114	0.62	3620	0.54	1.5	7.2	0.67	1.85	10.0	0.80	2.2	11.0	0.94	2.6	11.1
	1-2-2	3.58	126	0.85	3810	0.31	0.8	1.4	0.44	1.15	8.1	0.57	1.5	9.3	0.71	1.85	9.5
	1-2-3	4.41	125	1.07	3900	0.09	0.25	0.4	0.22	0.55	5.9	0.35	0.9	7.7	0.49	1.25	7.9
	1-2-4	5.27	125	1.28	4000				0.01	0.0	2.9X	0.14	0.35	6.1X	0.28	0.7	6.5
	1-2-5	6.14	125	1.49	4100										0.07	0.15	4.0
	1-2½-0	2.50	114	0.77	4520	0.39	0.85	0.5	0.52	1.15	8.1	0.65	1.45	10.5	0.79	1.75	11.0
	1-2½-3	4.79	127	1.10	4820	0.06	0.1	0.3	0.19	0.4	1.2	0.32	0.65	5.5	0.46	0.95	7.7
	1-2½-4	5.73	125	1.39	4910							0.03	0.05	3.0	0.17	0.35	5.5
	1-3-0	3.00	114	0.93	5420	0.23	0.4	0.1	0.36	0.65	1.7	0.49	0.9	8.0	0.63	1.15	9.2
	1-3-2	4.47	125	1.08	5620				0.21	0.35	0.6	0.34	0.6	4.0	0.48	0.85	7.4
	1-3-3	5.28	126	1.25	5710				0.04	0.05	0.2	0.17	0.3	1.1	0.31	0.5	4.9
	1-3-4	6.04	128	1.35	5810							0.07	0.1	0.3	0.21	0.35	3.1
Limestone	1-1½-2	3.01	126	0.71	2900	0.45	1.55	6.7	0.58	2.0	8.9	0.71	2.45	9.5	0.85	2.95	10.0
	1-1½-3	4.00	121	1.07	3000	0.09	0.3	1.9X	0.22	0.75	6.4	0.35	1.15	7.6	0.49	1.65	8.1
	1-2-2	3.50	125	0.85	3810	0.31	0.8	1.2	0.44	1.15	7.6	0.57	1.5	8.4	0.71	1.85	9.1
	1-2-3	4.37	124	1.09	3900	0.07	0.2	0.1	0.20	0.5	4.1	0.33	0.85	6.9	0.47	1.2	7.1
	1-2-4	5.34	121	1.42	4000							0.0	0.0	3.7X	0.14	0.35	4.9
	1-2½-3	4.76	126	1.12	4820	0.04	0.1	0.3	0.17	0.35	0.5	0.30	0.6	4.0	0.44	0.9	6.9
	1-2½-4	5.67	124	1.41	4910							0.01	0.0	1.0	0.15	0.3	3.8
	1-3-2	4.44	124	1.10	5620	0.06	0.1	0.1	0.19	0.35	0.3	0.32	0.55	3.3	0.46	0.8	7.0
	1-3-3	5.25	125	1.27	5710				0.02	0.05	0.1	0.15	0.25	0.7	0.29	0.35	4.0
	1-3-4	6.02	126	1.42	5810							0.0	0.0	0.0	0.14	0.25	1.9

Voids—(*v*)—Voids in dry rodded mixed aggregate in cu. ft. per sack of cement; computed from the values in the two preceding columns by means of the formula,

$$v = \frac{(165 - w)\,N}{165}$$

Surface Area—(*S*)—Surface area of aggregate in sq. ft. per sack of cement. The sieve analyses of the sand, gravel and limestone used in Series 212 are given in the report as follows, in terms of the percentage by weight larger than the given sieve size:

Sieve	No. 100	No. 48	No. 28	No. 14	No. 8	No. 4	⅜ In.	¾ In.	1½ In.
Sand	99	92	61	37	19	2	0		
Gravel and Stone	100	100	100	100	100	100	75	25	0

The grading of the coarse aggregates used in Series 186 is the same as that given above. No sieve analysis is given for the sand used in Series 186; it is here assumed that the grading is the same as that used in Series 212, as they are both from the same source of supply and have almost the same unit weights—113 lb. for Series 186 against 114 lb. for Series 212. Surface areas have been computed upon the assumption that the ratio of surface to volume for each individual particle is the same as that of a sphere, or $\frac{6}{d}$, where d is the average diameter in feet of the particles on each size sieve. The results thus obtained should be a fairly close approximation in the case of water-borne particles of typical sand deposits; it should be noted that exact values of the surface areas are not necessary provided that the values used are in some fairly constant ratio to the actual areas. This method of computation gives values of 1790 sf/cf for the sand in Series 186, 1810 sf/cf for the sand in Series 212, and 85 sf/cf for the gravel used in both Series. As variations in the surface area of the coarse aggregates are of little influence, the surface area of the limestone has been taken equal to that of the gravel.

Volume of Cement Paste—(*V*)—The sum of the volume of the mixing water plus the absolute volume of the cement, per sack of cement. Where x = the water-cement ratio, in terms of cu. ft. of water per sack of cement, $V = x + 0.49$.

Excess Cement Paste—(*V-v*)—The difference between the volume of the cement paste and the volume of the voids in the dry rodded mixed aggregate, expressed in cu. ft. per sack of cement.

Workability Factor—(*K*)—The quotient obtained by dividing the volume of excess cement paste by the surface area of the aggregate, and multiplying by 10,000 to provide a more convenient location for the decimal point. K therefore represents the average thickness of the cement paste film in ten-thousandths of a foot.

Slump—Taken directly from Fig. 8, 9, 15 and 16 of the report, and expressed in inches.

It is important to note at this point that slump is not an entirely satisfactory measure of the workability of the mix. It has been used for this purpose in Table 1 because, while both slump and flow are given in the report, there are more determinations of the former, and it is the more generally known function of the two. The slump test, however, has one serious defect; it gives misleading results where

segregation occurs, as in the case of undersanded sizes. In this connection, the language of the report is important:

> Harshness occurred in mixes that were undersanded. In such cases the slump test was of little value in determining workability because of the crumbling that sometimes took place when the cone was removed. The slump of a mix of this type might vary from almost nothing up to 6 or 7 inches, depending upon how the cone was removed.
>
> Segregation took place when the slump cone was removed and on the flow table in extremely wet consistencies of both lean and rich mixes. It was more pronounced in the harsh than in the oversanded mixes. Mixes in which the ratio of coarse to fine was 2 or more were found to be particularly liable to segregate. While a wet harsh mixture naturally gave a large slump, this did not necessarily indicate true workability, the general appearance of the mix being the best indication.

Certain values of the slump in Table 1 will be found marked with the letter X. These are values which, upon plotting, are found to be out of line with the rest of the data, in each case the slump being considerably higher than the bulk of the results. On investigation, it will be seen that all of these results are from mixes in which the ratio of coarse to fine was 2 or more; of the 10 mixes in question, 4 were 1-1½-4, 2 were 1-1½-3, 3 were 1-2-4, and 1 was 1-2-5. All are therefore within the range of undersanded mixes according to the wording of the report; they have therefore been ignored in plotting the values of Table 1.

Table 1 contains all the workable mixes which can be obtained from Fig. 8, 9, 15 and 16 of the report, only a few in which the volume of cement paste is less than the volume of the voids being neglected. With the exception of the undersanded mixes discussed in the preceding paragraph, and with the further exception of the straight mortar mixes (1-1-0, 1-1½-0, etc.) which are included elsewhere, all the values of Table 1 have been plotted in Fig. 3, the slump in inches being plotted against the workability factor (K) for each of the four water-cement ratios. The four separate curves of Fig. 3 have been combined into one diagram in Fig. 4.

In considering the variations of individual plotted points from the curves, several factors must be considered. An extreme range of mixes is here represented. The two series of tests were made more than two years apart; no information about the cement is available except that in each case it was a mixture of four brands on the Chicago market. Two different coarse aggregates were used,—gravel and broken stone. The grading of the sand cannot have been exactly the same in the two Series, as is shown by small differences in unit weight for identical mixes. The slump test is not in itself a precise measurement, slight variations in handling affecting the results con-

siderably. The determination of the quantity V-v is subject to the inaccuracy which applies to all residual quantities. Finally, the shape of the basic curves from which the necessary interpolations were made must be taken into consideration; due to the characteristic reversed curve, more than three points may be necessary to permit of satisfactory interpolation, and many of the curves (especially those of Series 212) have no more than three points determined by experimental values. The actual variations from the mean curves in Fig. 3 are within the limits expected from an analysis of the experimental data on which they are based.

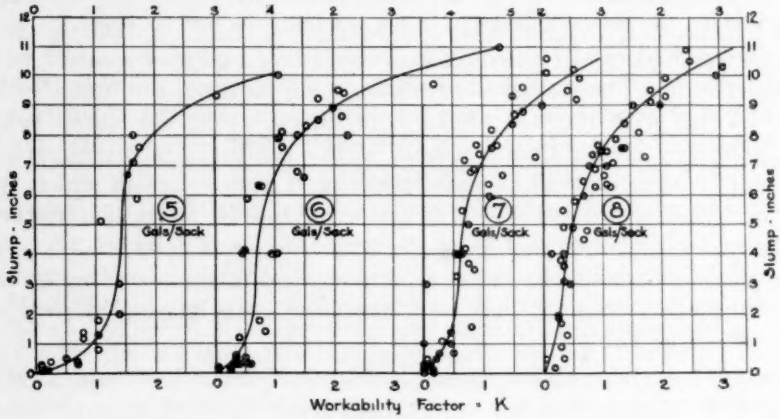

FIG. 3

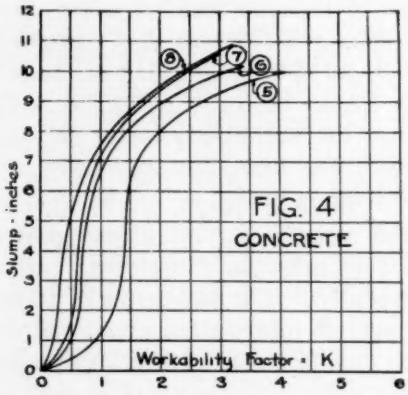

FIG. 4

The data given in Table 6 of the report are subject to fewer uncertainties. This table summarizes the results of the tests in Series 213, in which only straight mortar mixes were used. Instead of varying

proportions of fine and coarse aggregates, seven different gradings of sand were used, with unit weights and sieve analyses as follows:

Mark	Size	Unit Weight	Sieve Analysis					
			No. 100	No. 48	No. 28	No. 14	No. 8	No. 4
A	0–14	104	95	85	30	0	0	0
B	0–8	108	96	88	44	20	0	0
C	0–4	115	97	90	54	24	15	0
D	0–4	116	98	93	67	53	32	10
E	0–¾	118	99	96	83	75	55	25
F	20–30	104	100	100	66	0	0	0
G	20–30	98	100	100	66	0	0	0

It will be noted that these sands cover a wide range of sizes. A to E in order range from very fine to unusually coarse, while F and G represent standard Ottawa sand and a local sand of the same artificial grading.

The tests in Series 213 are arranged in sets, each of a constant water-cement ratio. This uniformity is not merely nominal, as in Series 186 and 212; the proper correction for absorption was made for each mix, enough additional water being added to maintain the same true water-cement ratio. It is therefore possible to plot the data directly, without the necessity of interpolation.

Table 2 herewith represents the data derived from Table 6, Series 213, of the report, including water-cement ratios from 4½ to 7 gal. per sack. The same notes apply, in general, as given for Table 1 above. The data from Table 2, together with the mortar mixes of Table 1, are plotted in Fig. 5. The six curves of Fig. 5 are reproduced in Fig. 6.

It will be noted that, for all mixes of Series 213, the volume of cement paste is in excess of the volume of voids in the aggregate. The first criterion of workability is therefore satisfied for all mixes. No observations as to workability are included for this Series in the report; instead, mixes have been arbitrarily classified as workable or non-workable according to the slump, the non-workable mixes being those with slumps of less than 1 in.

As was to be expected from the nature of the data, the plotted points of Fig. 5 and the family of curves of Fig. 6 show a high degree of regularity, and offer striking proof of the workability factor method of design.

A comparison of the curves for mortars in Fig. 6 with those for concretes in Fig. 4 brings out several interesting points. As the water-cement ratio increases, the curves approach each other; that is, the effect of the consistency of the cement paste becomes of less

TABLE 2

Mix	Sand Size	Voids v	Surface Area S	4½ gal. per sack $V = 1.09$			5 gal. per sack $V = 1.16$			5½ gal. per sack $V = 1.22$			6 gal. per sack $V = 1.29$			6½ gal. per sack $V = 1.36$			7 gal. per sack $V = 1.42$		
				$V-v$	K	Slump	$V-v$	K	Slump	$V-v$	K	Slump	$V-v$	K	Slump	$V-v$	K	Slump	$V-v$	K	Slump
1-1	A	0.37	2300	0.72	3.15	9.2	0.79	3.35	10.8												
	B	0.34	2070	0.75	3.6	9.6															
	C	0.30	1970	0.79	4.0	10.4															
	D	0.29	1490	0.80	5.35	10.0															
	E	0.28	980	0.81	8.25	10.2															
1-1½	A	0.56	3470	0.53	1.5	4.5	0.60	1.75	8.7	0.66	1.9	10.1									
	B	0.52	3100	0.57	1.85	6.0	0.64	2.05	9.5												
	C	0.46	2950	0.63	2.15	8.5	0.70	2.35	10.0												
	D	0.45	2230	0.64	2.85	8.4	0.71	3.2	10.2												
	E	0.43	1470	0.66	4.5	9.4	0.73	4.95	10.4												
	F	0.56	2310	0.53	2.3	9.2	0.60	2.6	10.2												
	G	0.61	2180	0.48	2.2	6.8	0.55	2.5	9.6												
1-2	A	0.74	4600	0.35	0.75	0.4	0.42	0.9	1.3	0.48	1.05	5.7	0.55	1.2	9.0	0.62	1.35	10.0			
	B	0.69	4140	0.40	0.95	1.1	0.47	1.15	3.3	0.53	1.3	7.8	0.60	1.45	9.5						
	C	0.61	3940	0.48	1.2	2.0	0.55	1.4	7.2	0.61	1.55	9.6	0.68	1.75	10.1						
	D	0.59	2980	0.50	1.7	5.0	0.57	1.9	8.5	0.63	2.15	10.4									
	E	0.57	1960	0.52	2.65	8.1	0.59	3.0	9.9												
	F	0.74	3080	0.35	1.15	5.0	0.42	1.35	8.0	0.48	1.55	9.1									
	G	0.81	2900	0.28	0.95	1.0	0.35	1.2	3.7	0.41	1.4	7.6									
1-2½	A	0.93	5800							0.29	0.5	0.7	0.36	0.6	1.5	0.43	0.75	4.9	0.49	0.85	8.0
	B	0.87	5170							0.35	0.7	1.4	0.42	0.8	4.7	0.49	0.95	7.4	0.55	1.05	9.2
	C	0.76	4920				0.40	0.8	1.0	0.46	0.95	5.4	0.53	1.1	8.3	0.60	1.2	9.9			
	D	0.74	3720				0.42	1.25	1.5	0.48	1.3	8.0	0.55	1.5	9.2	0.62	1.7	9.6			
	E	0.71	2450	0.38	1.55	3.0	0.45	1.85	7.6	0.51	2.1	9.2	0.58	2.35	10.0						
	F	0.93	3850				0.23	0.6	1.2	0.29	0.75	2.4	0.36	0.95	6.4						
	G	1.03	3620				0.13	0.35	0.6	0.19	0.55	1.2	0.26	0.7	2.6						
1-3	A	1.11	6960													0.25	0.35	0.9	0.31	0.45	1.8
	B	1.04	6200													0.32	0.55	1.2	0.38	0.60	2.0
	C	0.91	5900										0.38	0.65	1.6	0.45	0.75	4.8	0.51	0.85	8.4
	D	0.86	4460										0.40	0.9	6.0	0.47	1.05	8.2	0.53	1.2	9.6
	E	0.86	2940							0.36	1.25	7.0	0.43	1.45	5.6	0.50	1.7	9.2			
	F	1.11	4620										0.18	0.4	0.8				0.31	0.7	2.6
	G	1.22	4350										0.07	0.15	0.1				0.20	0.45	1.4

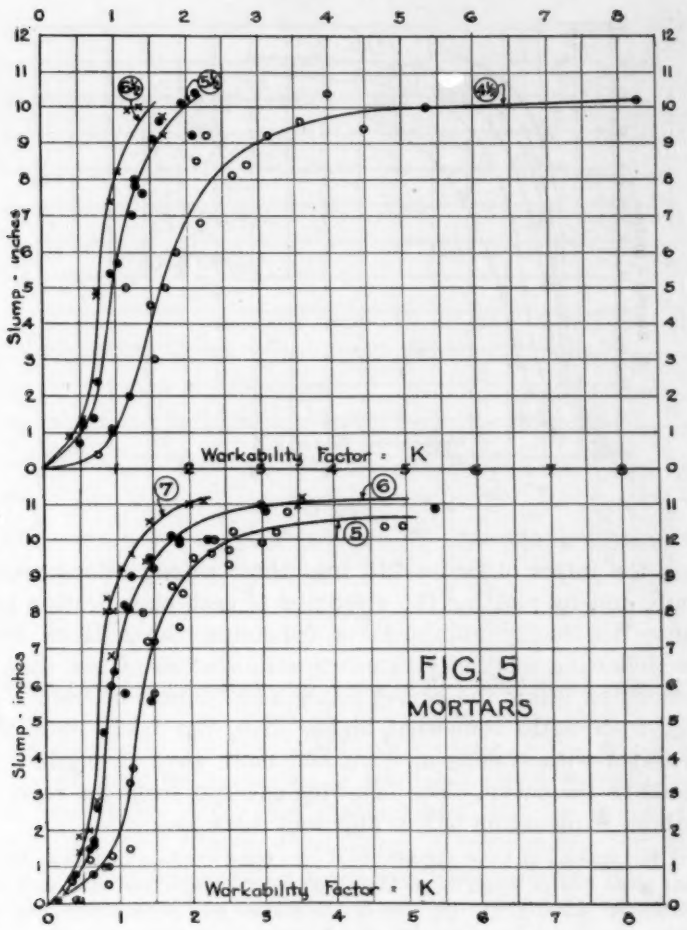

FIG. 5

importance. This tendency, shown very clearly in Fig. 6, explains why the plotted points for 7 and 8 gal. per sack are so close in Fig. 3. When the curves for identical water-cement ratios in the two sets of diagrams are compared, it will be seen that the curves are of the same distinctive shape, but that those for mortars are uniformly higher, except for small values of K, than those for concrete.

The authors of the report have noted a similar phenomenon in the strength relationship of mortars and concretes. In plotting the water-cement strength curves for the mixes of Series 213, it was found that the results ranged considerably higher (500 to 800 p.s.i.) than the curve previously obtained from the concrete series. By

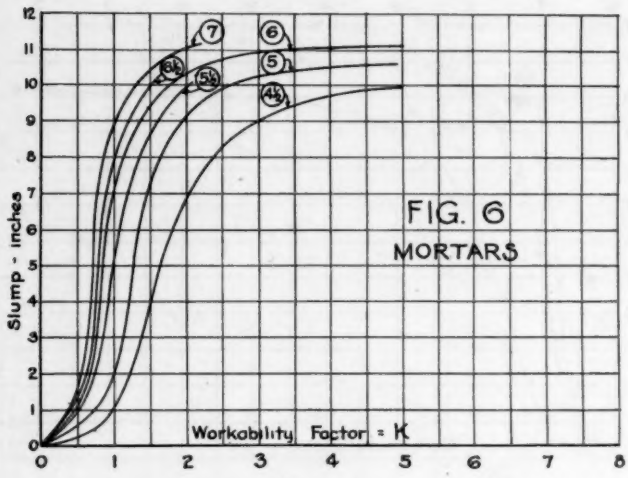

FIG. 6

dividing the mixes of Series 213 into three classifications according to slump, and by plotting the strengths of each classification against the curve previously obtained for concretes, the authors brought out the following relationship: mixes with slumps of less than 1 in. gave strengths below the curve; mixes with slumps of from 1 in. to 4 in. gave strengths somewhat higher than, but fairly close to, the curve; mixes with slumps of 5 in. and more gave strengths considerably above the curve. The following excerpts from the report give the authors' explanation of this apparent discrepancy.

During the making of these specimens, it was observed that in the wetter mixes there was some loss of water in molding due to leakage of the mold and also an accumulation of water at the top due to the greater ease with which these wetter mixes segregate.....Even where water is not lost directly from the specimen, the segregation which brings excess water to the surface changes the water ratio in the body of the cylinder.....That somewhat higher strengths were obtained in these mortars than with the concretes of previous Series for the same water ratios appears to be largely explained by the reduced water-cement ratio resulting from segregation and loss of water in the specimen.

The effect of segregation, as was noted in the case of undersanded mixes, is to increase the slump. Therefore, the greater tendency towards segregation noted by the authors in the case of the wetter mortar mixes may be taken as an explanation of the larger slumps obtained for the same values of the workability factor, K.

An interesting and important relation is demonstrated in Fig. 7. The fundamental purpose of the tests in Series 186 of the report (as in the earlier Series 122) was to determine the effect of workability

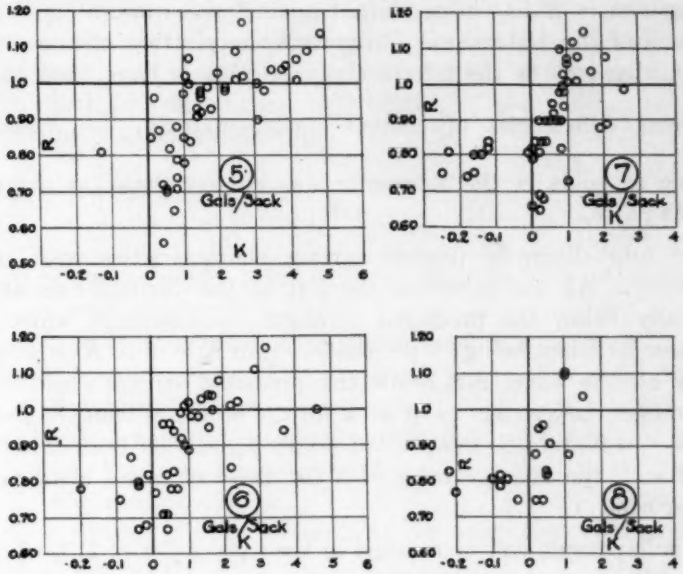

FIG. 7

upon the water-cement ratio strength law. The result was to emphasize the limitation set forth by Abrams in his original statement of the law,—namely, the law holds only as long as the mix is workable. By plotting the strengths of workable mixes separately from those of unworkable mixes, the authors show that the former follow a curve of the same type as the original Abrams curve, but slightly higher in value. The strengths of the unworkable mixes, however, fall considerably below this curve. It will be remembered that the authors' classification of workable and non-workable mixes was based upon very careful and thorough observation.

The curve given by the authors for the workable gravel mixes of Series 186 corresponds to the equation $S = \dfrac{16{,}000}{\overline{6.3}^{x}}$. The predicted strengths according to this formula have been computed for each of the 150 gravel mixes of Series 186, and a value R has been obtained for each mix by dividing the actual reported strength by the predicted strength. The tests have been divided into four groups,—5, 6, 7 and 8 gal. per sack,—each test being classified with the nearer of the four water-cement ratios previously used in Figs. 3 and 4. A separate diagram is shown for each group in Fig. 7, in which the

strength ratio R has been plotted against the workability factor K for each of the 150 mixes. To permit the plotting of non-workable mixes, abscissae to the left of the vertical axis have been taken to represent deficiencies of cement paste,—namely, $\frac{v\text{-}V}{v}$, where the volume of voids in the aggregate was greater than the volume of cement paste.

The four diagrams present certain characteristics with striking regularity. All the mixes to the left of the vertical axis are substantially below the predicted strength,—the average value of R for these 25 mixes being 79 per cent. From $K = 0$ to $K = 0.9$, practically all the mixes are below the predicted strength and cover a wide range, the average value of R for the 60 mixes being 84 per cent. For $K = 1.0$ and up, the plotted points appear on both sides of the line $R = 1$, the average value of R for these 65 mixes being actually 100 per cent.

A similar result will be recalled in the case of Series 213, where the mixes with a slump of less than 1 in. were considerably below the others in strength. From Fig. 6, a 1 in. slump corresponds to values of K from 0.3 to 1.0, depending upon the w/c ratio.

The diagrams in Fig. 7 demonstrate the validity of the writer's corollary to the water-cement ratio law, namely: the water-cement strength relationship applies only to mixes in which the volume of cement paste is sufficient to (1) fill the voids in the dry rodded mixed aggregate and (2) provide an excess of paste proportional to the surface area of the aggregate. But the observations of the authors of the report definitely limit the application of the law to workable mixes. Therefore the criteria (1) and (2) above are the true measure of the workability of the mix.

THE DESIGN OF CONCRETE MIXES

In a preceding article the following laws of workability were demonstrated:

For a mix to be workable, the volume of cement paste must be at least equal to the volume of the voids in the dry rodded mixed aggregate. For any required degree of workability, the necessary excess amount of cement paste depends (a) upon the consistency of the cement paste itself,—lower water-cement ratios requiring larger excess amounts than higher ones,—and (b) upon the surface area of the aggregate,—the larger the surface area, the greater the excess required.

For a fixed water-cement ratio, the workability of the mix is seen to depend upon the workability factor, K, which is defined as the excess cement paste divided by the surface area of the aggregate,—and multiplied, for convenience, by 10,000.
The following equations then apply:

$$x + a = N\left[\frac{w_s - w}{w_s} + \frac{KS}{10{,}000}\right] \quad \text{or} \quad N = \frac{x + a}{\frac{w_s - w}{w_s} + \frac{KS}{10{,}000}}$$

where x = water-cement ratio in terms of cubic feet of water per sack of cement
a = absolute volume of cement in cubic feet per sack
w_s = specific weight of aggregate in pounds per cubic foot of absolute volume
w = unit weight of aggregate in pounds per cubic foot of dry rodded mixed aggregate
K = workability factor
S = surface area in square feet per cubic foot of dry rodded mixed aggregate
N = true mix in terms of cubic feet of dry rodded mixed aggregate per sack of cement

The surface area may be obtained from the sieve analysis, with sufficient accuracy for the purpose, by the formula

$$S = \frac{6w}{w_s}\Sigma\frac{p}{d}$$

where $\Sigma\frac{p}{d}$ represents the summation of the quotients obtained by dividing the proportion of the total weight of the sample retained on each sieve (p) by the average diameter of the particles retained on the same sieve (d). In this computation, the amount of aggregate passing the 100 sieve is neglected.

In addition to the physical constants of the materials, the complete design of the mix requires only the proper choice of the two variables, x and K, the water-cement ratio and the workability factor, respectively. For the first, reference must be made to the familiar Abrams curve, in which strength is plotted against water-cement ratio. For the second, a series of curves, of the type illustrated by Fig. 8, are required, in which the slump is plotted against the workability factor, K, for various values of the water-cement ratio. For both, in large and important work, job curves should be prepared for the particular materials to be used.

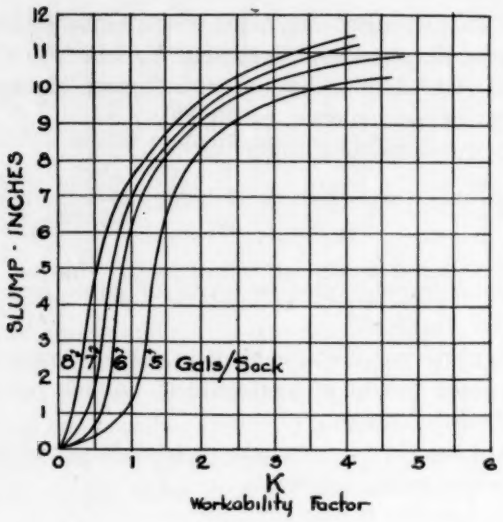

FIG. 8

The application of this method to the usual case, in which separate fine and coarse aggregates are available, is illustrated in Fig. 9. Let it be required to design a mix for six gallons per sack ($x = 0.80$) and a slump of 6 in., using fine and coarse aggregates of the following characteristics:

Aggregate	Unit Wt. w	100	48	28	14	8	4	⅜ In.	¾ In.	1½ In.
Fine.....	116	99	92	63	42	20	0	0	0	0
Coarse.....	110	100	100	100	100	100	100	75	34	0

The specific weight of the aggregate is taken as 165 lb. per cu. ft., corresponding to a specific gravity of 2.65; the absolute volume of a sack of cement is taken as 0.49 cu. ft., corresponding to a specific gravity of 3.10.

It is first necessary to determine experimentally the unit weight of the mixed aggregate for varying proportions of fine to coarse. Four mixtures are prepared, in which the proportion by weight of fine aggregate to total aggregate is respectively 0.30, 0.40, 0.50 and 0.60. The weight of the dry rodded mixed aggregate per cubic foot is determined for each mixture. These four results, together with the unit weight of the separate aggregates, are plotted on Diagram A, in which the abscissae throughout represent proportions by weight of fine aggregate to total aggregate in the mix. A smooth curve

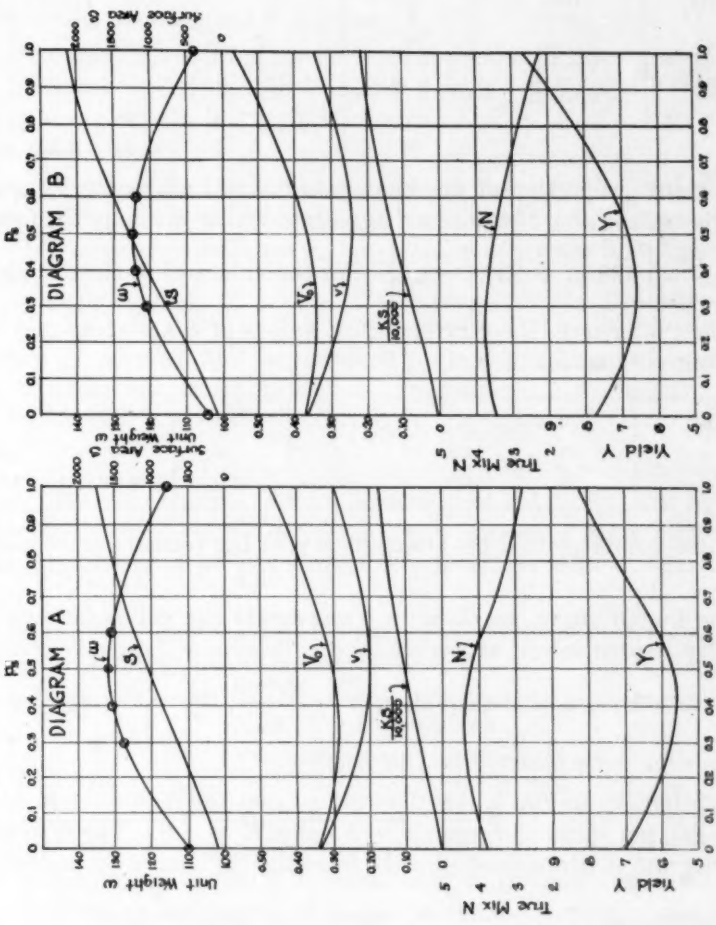

FIG. 9—(SEE PAGES 393, 394 AND FOLLOWING)

drawn through these six points gives the curve marked "w," which represents the variation in unit weight of the dry rodded mixed aggregate.

To determine a similar curve for surface areas, it is first necessary to compute the values $\Sigma \frac{p}{d}$ for both the fine and coarse aggregates, as follows:

		Fine Agg.		Coarse Agg.	
Screen	d	p	p/d	p	p/d
1½ in.	0.1875	0	0	0	0
¾ in.	0.0938	0	0	0.34	3.5
⅜ in.	0.0469	0	0	0.41	8.8
4	0.0234	0	0	0.25	10.7
8	0.0117	0.20	17	0	0
14	0.00586	0.22	38	0	0
28	0.00293	0.21	72	0	0
48	0.001465	0.29	198	0	0
100	0.000733	0.07	95	0	0
			420		23.0

For any proportion of fine aggregate to total aggregate by weight, p_s, the surface area of the mixed aggregate will be given by the formula $S = \frac{6w}{w_s}\left[420\ p_s + 23\ (1-p_s)\right]$. For the values of p_s already plotted on the "w" curve, the corresponding values of S are computed as in the following table:

p_s	0	0.3	0.4	0.5	0.6	1.0
w	110	128	131	132	131	116
S	90	660	870	1070	1240	1770

These points plotted on Diagram A give the second curve, marked "S".

The fourth curve, marked "v," represents the volume of voids in the dry rodded mixed aggregate, and is determined directly from the "w" curve by use of the equation, $v = \frac{w_s - w}{w_s}$, the value of w for each value of p_s being taken from the diagram.

By reference to Fig. 8, it is determined that the required 6 in. slump for 6 gal. per sack, corresponds to a value $K = 1.0$. The fifth curve on Diagram A is obtained directly from the "S" curve by multiplying by the constant factor, $\frac{K}{10{,}000}$, where K in this case equals 1.0.

The ordinates for both the "v" and $\frac{"KS"}{10{,}000}$ curves represent volumes in cubic feet per cubic foot apparent volume of dry rodded mixed aggregate; the "v" curve gives the volume of cement paste required to fill the voids in the dry rodded mixed aggregate, while the $\frac{"KS"}{10{,}000}$ curve gives the volume of cement paste required to coat the individual particles of aggregate and give the required degree of workability. The total volume of cement paste per cubic foot of dry rodded mixed aggregate is therefore obtained by adding corresponding values of the two curves; the third curve on Diagram A is obtained in this manner, and is marked "V_o".

The volume of cement paste per sack of cement is determined from the equation $V = x + aq$. For 6 gal. per sack, $x = 0.80$, and $V = 1.29$. The true mix is obtained from the equation $N = V/V_o$ as given by the previous curve.

The volume of freshly mixed concrete is equal to the sum of the absolute volumes of the materials. The yield is usually expressed in terms of sacks of cement per cubic yard of concrete; it is therefore given by the formula $Y = \frac{27}{x + a + Nw/w_s}$. The last curve on Diagram A, marked "Y", is obtained by inserting the proper values of N and w in the formula, $Y = \frac{27}{1.29 + Nw/165}$.

An inspection of Diagram A shows that the minimum cement content will be obtained with a true mix of 4.4 cf. of dry rodded mixed aggregate per sack of cement, the aggregate being combined in the proportion of 44 per cent of fine to 56 per cent of coarse, by weight.

Diagram B, Fig. 9, represents the same problem of design, with aggregates of the following characteristics:

Aggregate	Unit Wt. w	Sieve Analysis								
		100	48	28	14	8	4	⅜	¾	1½
Fine.....	108	98	88	44	20	10	0	0	0	0
Coarse.....	104	100	100	100	100	100	100	75	20	0

An inspection of Diagram B shows that, with these materials, the, minimum cement content will be obtained with a true mix of 3.8 cf., 33 per cent fine to 67 per cent coarse by weight.

The workability-factor method of design is of fundamental importance in its relation to many of the principal problems of the concrete mix.

Strength—The basic Abrams water-cement ratio law must be amended to read: The strength of concrete is determined by the amount of mixing water, as long as the volume of cement paste is sufficient to fill the voids in the dry rodded mixed aggregate and to provide an excess proportional to the surface area of the aggregate. In the following, references are again to Table 3 of the paper by McMillan and Johnson, reporting the results of the tests of Series 212, group 2.

By plotting strength against w/c ratio for all mixes noted as "workable" by the operator, a curve of the familiar type was obtained, from which the predicted strength could be interpolated for any given value of the w/c ratio. In the following table, the first three columns are taken directly from the report; the fourth gives the predicted strength obtained by interpolation from the w/c curve; the fifth column gives the value of R, the ratio of the actual strength to the predicted strength; the last column gives the value of the workability factor, K, obtained from the data presented in the report by the methods described above. This table represents only one family of mixes of the many given in the report, but it is typical of the results obtained.

Ref.	x = w/c	Strengths		R	K
		Actual	Predicted		
166	0.89	3530	3330	1.06	2.5
167	0.85	3620	3600	1.01	1.6
168	0.83	3690	3740	0.99	0.75
169	0.80	3630	3980	0.91	0.05
170	0.77	3280	4220	0.78	(deficient in cement paste)

The results for a similar computation for all the sand-and-gravel mixes of Series 212-2 are shown in Fig. 10, in which R is plotted against K. Points to the left of the vertical axis represent mixes deficient in cement paste, the ratio of the volumes of paste and voids being used as abscissae; all of these are below the predicted strength, the average value for the eight tests being 87.5 per cent. From $K=0$ to $K=1$, the bulk of the results are below unity, the average value for the twenty-three tests being 92.7 per cent. Where $K=1$ or more, the points are about uniformly divided above and below the line $R=1$, the average value for the twenty-one tests being 98.4 per cent. A comparison of this figure with Fig. 7, in which the same process was

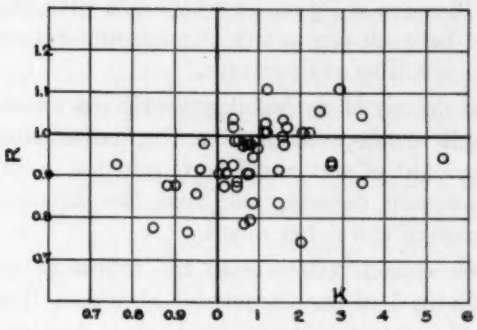

FIG. 10

applied to a different series of tests, emphasizes the importance of the workability factor in the strength relationship.

Economy—Each diagram of Fig. 9, taken by itself, indicates one aspect of the relationship of the workability factor method of design upon the important consideration of economy. The "Y" curve in each diagram permits the proper choice of proportions of fine and coarse aggregates to obtain the greatest possible economy with given materials.

From an inspection of the diagrams, it will be noted that the low point of the V_s curve occurs at a lower proportion of fine aggregate than the low point of the v curve; that is, the amount of cement paste required for a given degree of workability is a minimum at a mix coarser than that giving maximum density. This is a general relationship. The slope of the v curve varies from a negative at $p_s = 0$ to zero at the point of maximum density, and is positive from there on; the slope of the S curve is positive throughout. Since the V_s curve is a summation of the v and $\frac{KS}{10{,}000}$ curves, the slope of the V_s curve will be zero (and V_s therefore a minimum) at a point where the slope of the v curve is equal and opposite in sign to the slope of the $\frac{KS}{10{,}000}$ curve. Obviously this must occur at a point to the left of the low point of the v curve. Furthermore, since the slope of the $\frac{KS}{10{,}000}$ curve increases with K, the point where the slopes are equal

and opposite will move still further to the left with richer mixes, and the discrepancy between the points of minimum cement content and maximum density will be even greater.

With a given degree of workability and fixed amounts of cement and aggregates, it will be evident from the above considerations that the minimum amount of water will be required at a mix coarser than that giving maximum density, and that the difference will be increased in the case of the richer mixes.

Compare these considerations with the following quotations from the famous Bulletin 1 of the Structural Materials Research Laboratory—"Design of Concrete Mixes," by Duff A. Abrams.

> With given concrete materials and conditions of test the quantity of mixing water used determines the strength of the concrete, so long as the mix is of a workable consistency.
>
> * * *
>
> There is an intimate relation between the grading of the aggregate and the quantity of water required to produce a workable concrete.
>
> * * *
>
> The aggregate grading which produces the strongest concrete is not that giving the maximum density (lowest voids). A grading coarser than that giving maximum density is necessary for highest concrete strength.
>
> * * *
>
> The richer the mix, the coarser the grading should be for an aggregate of given maximum size; hence, the greater the discrepancy between maximum density and best grading.

The first two quotations are given to illustrate the context in the light of which the last two quotations must be read. When Mr. Abrams writes of the effect of the grading of the aggregate upon the strength of the concrete, the whole tenor of his paper makes it clear that he refers to the effect upon the amount of mixing water required for workability; that is to say, "the aggregate grading which produces the strongest concrete" should be read, "the aggregate grading which requires the least mixing water." An exact re-statement of the third and fourth quotations would be: For a given degree of workability and fixed amounts of cement and aggregates, the minimum amount of mixing water is required at a grading somewhat coarser than that giving maximum density; the richer the mix, the greater this difference. But this is exactly the same conclusion derived from the curves of Fig. 9. It is a striking proof of the validity of the workability factor theory, that it explains on rational grounds experimental data obtained in the development of an entirely different hypothesis.

Of more practical importance, from the point of view of economy, is a comparison of Diagrams A and B with each other. Combined in

the most economical proportions, the aggregates of Diagram A require 5.6 sacks of cement per cubic yard of concrete; the corresponding figure for the aggregates of Diagram B is 6.6 sacks. The difference between the "A" aggregates and the "B" aggregates therefore amounts to a full sack of cement per cubic yard of concrete.

The workability factor conception makes it possible to define for the first time what is meant by that much-abused term,—"a well-graded aggregate." A well-graded aggregate is one that combines the primary requirement of a low percentage of voids with the secondary requirement of a low surface area; these characteristics, together with a certain degree of continuity of grading (to eliminate freak "gap" gradings) insure an economical mix. This method also demonstrates the advantage of using as large a maximum size of aggregate as is practicable; lower percentages of voids are obtainable with larger aggregate, and the surface area is reduced.

Durability of Concrete. The use of a low cement content is not merely a matter of low first cost. Emphasis in recent years upon a high *quality* of cement paste has obscured the importance of a low *quantity* of paste. Of two mixes, of the same w/c ratio and degree of workability, the one containing the less cement paste per cubic yard will produce the better concrete.

It is the hardened cement paste,—the matrix,— that is vulnerable to the agents of deterioration. With even the driest concretes in field use today, probably fifty per cent of the mixing water is in excess of that required for the hydration of the cement. It is the evaporation of this excess water that is responsible for the initial shrinkage which accompanies the drying out of freshly-poured concrete and which is probably the greatest single factor leading to its eventual deterioration. Where retained in the concrete, this uncombined water destroys the homogeneity of the matrix and is a definite source of danger in case of freezing. Expansion and contraction due to changes in moisture are obviously characteristics of the matrix. Resistance to abrasion is not a quality associated with cement paste. In short, the porous nature of the matrix is closely associated with those characteristics of concrete which make for impermanence.

The function of cement paste in the freshly-mixed concrete is that of an internal lubricant, and a definite minimum amount is necessary; in the hardened concrete, its function is simply that of a binder holding the particles of aggregate together. As in a glued joint, no proportionate increase in strength is obtained by an increase in the amount of binder. The amount of cement paste should therefore be

held to the minimum consistent with workability, and the proper grading of the aggregates not only promotes economy but also contributes to the durability of the concrete. While definite data are lacking, there is some evidence that present day cements are responsible for a larger coefficient of shrinkage in the concrete; this tendency, if real, increases the importance of the proper design of concrete for minimum cement paste content.

Control—A striking characteristic of the workability curves is that, in the usable range, they are almost vertical. For slumps of from 2 in. to 6 in., small changes in the workability factor produce large changes in the slump. This explains the phenomenon familiar to every field worker in concrete,—the sudden variation in slump occurring without obvious change in the mix. The workability factor translates this rapid change of slope into an accurate method of control; provided that the grading of the aggregates remains constant, slight variation in the water-cement ratio are readily detected by the corresponding change in the slump.

The design of the mix according to the workability factor method requires only simple well-known laboratory determinations. The physical constants of the materials,—the specific weights (or specific gravities)—are easily determined, and vary very little for any given source of supply; if the nature of the aggregate is known, the specific weight may be obtained from any standard work of reference. In the case of the typical concrete aggregates,—silica and limestone,—the specific weights are practically identical, both being usually given as 165 lb. cu. ft. The determinations for unit weight and sieve analysis are both simple and reliable; the results are surprisingly uniform for aggregates of a common source. With reference to these two characteristics of the aggregate, it should be noted that, while the relationship between them is obscure, they are certainly not independent. For a given aggregate, as long as the sieve analysis remains the same, the unit weight will be constant. The converse, however, does not hold; a number of different gradings will produce the same unit weight. Finally, the slump test, if not demonstrably the most accurate measure of workability, is certainly the best known, and is best adapted to job conditions. Standard specifications for all of these tests have been available for years, and therefore no new technique is required in the application of the workability factor method of design.

Discussion to close in June 1940 JOURNAL *should reach A. C. I. Secretary in triplicate by Apr. 1, 1940.*

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Discussion of a paper by Charles T. Kennedy:

The Design of Concrete Mixes*

BY F. M. RUSSELL, JOHN SANFORD PECK, DUFF A. ABRAMS AND THE AUTHOR

Editor's Note: Convention discussion of mix design based on four papers (Charles T. Kennedy, February JOURNAL; R. F. Blanks, E. N. Vidal, W. H. Price and F. M. Russell, April JOURNAL; W. M. Dunagan, June JOURNAL; and Henry L. Kennedy, June JOURNAL) all having convention presentation in February 1940, is not readily assigned to any one of the papers in the published record of that discussion. Readers interested in the subject are referred not only to all four papers but to discussion (from the convention transcript and written contributions submitted subsequently) of all four concluded in this Supplement.

BY F. M. RUSSELL‡

Some use has been made in the past of the theory of proportioning concrete by computations based on supplying a quantity of cement paste in a concrete mix that is more than sufficient to fill the voids in the dry-rodded aggregate, and Archie A. Smith in particular carried the method to the point where he stated†:

> In general, the problem of proportioning ..... is narrowed to the determination of the value of T or T_m, as the case may be, that will actually produce a dense fresh mixture.

In this quotation the value of T is the thickness of the cement film that covers the surface of the sand particles and T_m is the thickness of the mortar film that covers the coarse aggregate particles after all the voids in the dry-rodded aggregate have been filled. The author's method progresses further than that advanced by Mr. Smith in that it has been developed to include the consideration of workability.

*JOURNAL, Amer. Concrete Inst., Feb. 1940, *Proceedings* Vol. 36, p. 373.
‡Assistant Engineer, Bureau of Reclamation, Denver, Colo. Mr. Russell participated in the convention discussion but replaced those remarks with this written contribution.
†"Proportioning—Its Fundamental Principles," Archie A. Smith, *Concrete*, May 1936.

Upon first studying the author's paper, it was interpreted that Fig. 8 was generally applicable to all aggregates and only in large and important work would it be necessary to construct a special set of curves. With that in mind, the writer applied the method to an actual job with the same fine and coarse aggregate gradings that were used in the work. The dry-rodded unit weights of different coarse-to-fine ratios were determined in accordance with A.S.T.M. designation C 29-27 and computations were performed according to the method set forth by the author. The water-cement ratio, by weight, was fixed at 0.57 which is equivalent to 6.4 gal. per sack of cement. A value of K was selected from Fig. 8 of the paper which, for a 4-in. slump and 6.4 gal. per sack, was found to be about 0.74. After going through the computations, the final mix was found to be 1:5.9, by weight, with a high gravel-to-sand ratio of 2.20 and a cement content of 1.45 bbl. per cu. yd. This is not in agreement with the mix actually used on the job which was 1:6.8, by weight, with a 1.61 gravel-to-sand ratio and a cement content of 1.33 bbl. per cu. yd. The mix, as determined by Kennedy's method, would give a slump of about 9 in. at the selected water-cement ratio of 0.57, by weight, instead of the desired 4 in.

It was concluded from these results that values for K must be determined for each different given set of materials and that a set of universal curves similar to those shown in Fig. 8 would not be applicable to all materials and hence a series of trial mixes must necessarily be made before a suitable value of K could be chosen. These results are presented merely to clarify this point in the paper for other readers who may have interpreted it as this writer did.

In the above application of the author's method, the sand was graded so that 5 per cent passed the 100 screen but was retained on the 200 screen. The surface area of this material amounted to 20 per cent of the total surface of the sand or more than the surface areas of the 8, 14, and 28 screen size materials combined. The amount passing the 100 screen was neglected in both the writer's and author's computations although it appears that such a large percentage of surface area should not be completely ignored.

The workability factor method of mix design is essentially a trial-mix method. The trial-mix method itself does not require the mathematical processes proposed in the author's paper for final selection of a satisfactory mix; however, the procedure proposed by the author might be used in analyzing a selected mix and in the evaluation of benefits to be derived from grading improvements by processing, blending, etc., although the less involved a method of mix design, the more practical it becomes.

The sheet of curves obtained by the author's method of analysis as represented by diagrams A or B present an excellent picture of the manner in which the given ingredients fit together.

The unit weight of dry-rodded aggregate is by no means the maximum unit weight that can be obtained or the unit weight that will probably apply after the aggregate is mixed into concrete and the concrete compacted in place. In a recent laboratory test the average unit weight of a dry-rodded mixed aggregate was found to be 129.2 lb. per cu. ft., whereas the unit weight of the same aggregate placed in two lifts instead of the usual three lifts, with each lift vibrated for 20 seconds, was found to be 136.0 lb. per cu. ft. In mixtures up to 1½ in. maximum, a workable concrete cannot be obtained with paste less than that required to fill the experimentally determined voids in the dry-rodded aggregate; however, it is entirely possible to compact 1½-in. maximum concrete mixes having no excess paste based upon the vibrated unit weight of 136.0 lb. per cu. ft. According to the author's paper, the strengths of unworkable mixes are considerably below those having sufficient paste. The "no excess paste" mix mentioned above was by no means considered workable, but when properly placed by vibration gave a compressive strength that checked within one per cent of the concrete containing the greater paste content of the same water-cement ratio. It is impracticable to make mixes using 1½-in. maximum aggregate without an excess of paste although it is not uncommon to find good workable mass concrete mixes (6-in. maximum aggregate such as is used in large concrete dams) which have less than sufficient paste to fill the experimentally determined voids in the dry-rodded aggregate. Such mass mixes have been used on several important jobs with vibration compaction and have given very satisfactory results.

Mr. Kennedy has made a noteworthy contribution in his presentation of the workability factor method of mix design. The workability factor, nevertheless, is a tool to be used with caution and judgment because its requirements vary widely for different conditions and methods of concreting and is dependent upon the opinion and experience of the observer.

BY JOHN SANFORD PECK*

There are two distinct schools of thought regarding the design of concrete mixes. The first, which might be called the "theoretical" school, holds that the design of concrete mixes is an exact science, and that a precise formula can be derived which will predict the strength of various mixes with mathematical accuracy. The second, which

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might be called the "pragmatic," or "empirical" school, holds that concrete is by its very nature such a variable material and there are so many variables involved in the design of any mix that mathematical accuracy and formulae are neither practical nor desirable. These empiricists hold, furthermore, that the law of diminishing returns is on their side and that the results do not warrant the time and trouble involved in applying any complicated theory.

Mr. Kennedy's paper places him definitely within the fold of the theoretical school. It is both scientific and scholarly and makes a definite contribution to what F. R. McMillan has called the "philosophy of concrete." The author has been able to apply his theory so as to obtain quantitative measures for some of the peculiar properties of concrete mixes which have only been measured qualitatively heretofore. Whereas McMillan and Johnson in their report of 1928 classified their mixes as either "workable" or "nonworkable" merely by the visual inspection of experienced experts, the author's method produces a "workability factor" with a definite value for each mix. This type of investigation is always a distinct contribution in any field, as it tends to increase our understanding of the phenomena involved.

But the writer gazes on the use of the workability factor directly in the design of a mix as explained by the author with the cold eye of an empiricist. The author states a "second law of workability—for any required degree of workability, the necessary excess of cement paste depends, (a) upon the consistency of the cement paste itself, and (b) upon the surface area of the aggregate." This law is all right as far as it goes, but it should include another important factor, namely, the shape of the particles comprising the coarse aggregate. A sphere will roll more readily than a cube, and hence a quartz pebble would require less lubrication to produce a given degree of workability than would a jagged particle of crushed limestone aggregate. The "workability factor" does not take this variable into account. This is just one example of how impossible it is to develop an all-inclusive theory.

In the actual design of a mix, the writer can see no advantage in choosing the "workability factor" instead of the slump, and he can see a great disadvantage in the work involved in obtaining the necessary data, making the sieve analyses, obtaining the weight per cubic foot of various mixtures of coarse and fine aggregate, computing the surface area, the true mix, and the "workability factor."

Those who deal with concrete continually, both on the job and in the laboratory, know that the probable error of any slump determination

is never less than one half inch plus or minus, and that a variation in the strength of any set of test cylinders of ten per cent plus or minus is not only allowable but usual. McMillan himself showed that the compressive strengths of various brands of cements as measured by the standard 1:3 mortar varied more than fifty per cent from the mean.

What is the advantage of trying to use precise methods with a material which exhibits such wide variations under the best of controlled conditions? The law of diminishing returns still operates and a smart man solves a problem the easy way. Nothing has been produced to date to compete in ease of operation and reliability with the McMillan revision of the Abrams Theory, which the concrete empiricists call the "trial batch method."

BY DUFF A. ABRAMS*

Introduction

Any attempt to clarify concrete mix design should be welcomed by the entire profession, and especially by those who have worked in this field. Many of the complications of this subject have arisen from attempts to deal with concrete by considering the "mortar" and "coarse aggregate" as separate entities. Arbitrarily splitting the aggregate into "fine" and "coarse" has only increased the difficulties. This is much like attempting to study animal behavior by first cutting off the animal's head. The aggregate must be considered as a whole. I am glad that Mr. Kennedy did not cut off the animal's head.

There seems to be a tendency to strive for too great refinement. With the utmost control of the characteristics of the constituent materials and the greatest accuracy in proportioning quantities, the resulting concrete is far from uniform. Concrete making is not yet an exact science. In a recent specification for a large concrete dam now under construction by the U. S. Bureau of Reclamation there are 64 instances in the *concrete section alone* where the final decision as to procedure lies in the "opinion of the contracting officer."

Mr. Kennedy has developed an approach to concrete mix design which deserves serious consideration. I do not agree with some of his assumptions or with all his conclusions; this however does not detract from an appreciation of his efforts to clarify this subject.

In the discussion which follows it has been found necessary to refer to certain of my concrete researches which were carried out at Lewis Institute, Chicago, as long as 25 years ago.

Water-Cement Ratio Law

The author states (p. 396):

*Consulting Engineer, New York.

The basic Abrams water-cement ratio law must be amended to read: The strength of concrete is determined by the amount of mixing water, so long as the volume of cement paste is sufficient to fill the voids in the dry rodded mixed aggregate and to provide an excess proportional to the surface area of an aggregate.

The following statement in Bulletin 1 of the Structural Materials Research Laboratory, Lewis Institute (1918), showed that the strength of the concrete depended on the *quality* of cement paste, within the limitations of *workable mixes:*

With given concrete materials and conditions of test the quantity of mixing water used determines the strength of concrete, so long as the mix is of a workable consistency.

Mr. Kennedy's "amendment" adds nothing tangible to this.

Mr. Kennedy's statement seems to imply that minimum voids in the dry rodded aggregates is the optimum condition for concrete mixes. Our work at Lewis Institute as early as 1915-18 showed this concept to be fallacious. Whatever the voids in the dry rodded aggregates it is necessary to destroy this condition before we can produce a workable concrete. Bulletin 1 pointed out:

The aggregate grading which produces the strongest concrete is not that giving the maximum density (lowest voids). A grading coarser than that giving maximum density is necessary for highest concrete strength.

Minimum Voids in Aggregates

Mr. Kennedy does not follow his principal of minimum voids to its logical conclusion, in that he stops short of suggesting a wide gap in aggregate grading. Many experimenters (beginning with Feret 50 years ago) have shown that if the total aggregate is divided into 3 size ranges which we may designate as fine (F), medium (M) and coarse (C), minimum voids will be obtained by mixing about 40 per cent F, 0 per cent M, and 60 per cent C. In a concrete sand this means:

F (0-50).....40 per cent
M (50-16)..... 0 per cent
C (16- 4).....60 per cent

For an aggregate graded to 1½ in. it means:

F (0-30).....40 per cent
M (30- 4)..... 0 per cent
C (4-1½).....60 per cent

Mr. Kennedy does not suggest this type of grading since it is obviously impracticable and is not compatible with the use of area of aggregates as a basis of mix design.

Area of Aggregates as a Basis of Concrete Mix Design

The concept of area of aggregates as a factor in proportioning concrete mixes is very old. Gillmore's book "Cements and Mortar,"

1863, suggested area as a basis for choice of sands. Several attempts have been made to use this factor as a rational basis for mix design; probably the most notable of these are to be found in the papers of Edwards and Young (*Proc.* A.S.T.M., 1918 and 1919). The Texas Highway Department made an elaborate experimental study of the surface area method (1931-2), but concluded it did not furnish a proper basis for mix design; see "Study of Portland Cement Concrete Aggregates."

Everyone who has attempted to use areas as a basis for mix design has been compelled to make compromises on one or more of the following features:

(1) Disregard the surface area of the cement.

(2) Neglect the area of the finest particles of sand.

(3) Assume particles as spherical.

(4) Recognize the fact that a minimum surface area of aggregate is not compatible with other known requirements.

Certain of these topics are discussed more fully below.

Computation of Aggregate Areas

Mr. Kennedy computed the surface area by making a number of assumptions, some of which have been mentioned; the following will be considered here:

(a) Neglect the surface area of 0 - 100 sand.

(b) Compute surface as spheres.

(c) Assume each size as represented by the average diameter of the group.

Neglect of 0 - 100 sand. In the example chosen by Mr. Kennedy the neglect of the area of the 0 - 100 sand made little difference, since there was only 1 per cent of this size; however this size may reach 5 to 10 per cent. Each 1 per cent of the 200 - 100 size gives an area equal to 4 to 6 per cent of the total, so that the inclusion of this size may increase the computed area up to 50 or 60 per cent; if the portion passing the 200 sieve were considered, the computed area would be further increased.

Computed as Spheres. The actual area will generally be much larger than that computed from this assumption. Most sands are sub-angular to angular in shape; in the case of crushed stone the actual area will probably be double that computed on the basis of spheres.

Average Diameter. Mr. Kennedy states that he uses the "average" diameter of each size group as a basis of computing area; however it seems that his figures are closer to the mid-diameter. Since the sieve sizes advance by multiples of 2 it is seen that in a single size group

there are 8 times as many of the smallest particles (in a pound) as of the largest, so that there is a vast difference between the *mid-diameter* of a group and the diameter of the *average particle*. The actual area may be double that computed on the basis of the mid-diameter.

It is only by courtesy that we can speak of the method used by Mr. Kennedy as giving the surface area of an aggregate; the actual area of a given sand may be 2 or 3 times that computed by his method. If surface area is significant, it will be necessary to get closer to the actual area than is possible by the method used.

We have already referred to the practice of neglecting the surface area of the cement, although modern test methods enable us to secure an approximate measure of this factor.

Fineness Modulus of Aggregate

The function which I suggested in Bulletin 1, 1918, known as the *fineness modulus* of aggregate has the practical effect of taking care of the requirements for grading of aggregates in a simple and practical manner; especially when we are dealing with the case of concrete aggregates which have been separately prepared to meet grading specifications. The fineness modulus was defined as the sum of the cumulative percentages of aggregates retained on each of the standard sieves, divided by 100.

Mr. Kennedy's statement that this "arbitrary function" "has been found to be demonstrably unsound" calls for discussion. With reference to my "Design of Concrete Mixtures" he states (p. 373):

> This presented two new concepts: (a) the water-cement ratio law, and (b) an arbitrary function of the grading of the aggregate known as the "fineness modulus."
>
> Experience has demonstrated the essential soundness of the water-cement ratio law. Not only the strength, but other principal characteristics of the concrete, have been found to be primarily dependent upon the ratio of the amount of mixing water to the amount of cement. The fineness modulus has not fared so well. It has been found to be demonstrably unsound, and has generally been abandoned.

Continuing the same discussion Mr. Kennedy states (p. 375):

> It is obvious that the choice of the proper water cement ratio is only the first step in the design of the mix. For a complete method of design, some means must be provided to determine the workability of the mix with regard to the amounts and grading of the aggregate. It was for this purpose that the fineness modulus was developed. Since the abandonment of this method, nothing has been advanced to take its place; even the Portland Cement Association has nothing better to offer than the mixing of trial batches—the method of "cut and try."

It is apparent that Mr. Kennedy's paper hinged to a large extent on his conclusion that the fineness modulus method was unsound; if that method had been considered sound, there would have been little point in writing the paper under discussion.

Methods of Computing Fineness Modulus

The fineness modulus may be computed in several ways; the simplest and most direct (using the sand in Mr. Kennedy's paper) is given in Table 1.

TABLE 1

Sieve No.	Cumulative Fraction Coarser than Sieve
100	.99
50	.92
30	.63
16	.42
8	.20
4	0
Fineness modulus	3.16

If the fractions are expressed in per cents, the sum must be divided by 100 in order to secure the FM.

In considering concrete aggregates, each group, as separated by the standard sieves, is treated as a single size represented by the mid-diameter of that group. From this viewpoint the FM of the above sand may be computed as in Table 2.

TABLE 2

Size Group			Separate Fractions of Sand	Fineness Modulus of Sand
Sieve No.	Mid-Diam.-in. (Log. Scale)	Fineness Modulus		
200—100	.0041	0	.01	0
100—50	.0082	1.00	.07	.07
50—30	.0164	2.00	.29	.58
30—16	.0328	3.00	.21	.63
16—8	.066	4.00	.22	.88
8—4	.131	5.00	.20	1.00
4—⅜	.262	6.00	——	——
⅜—¾	.525	7.00	——	——
¾—1½	1.05	8.00	——	——
1½—3	2.10	9.00	——	——
Sum	——	——	1.00	3.16

Columns 2 and 3 show that the size of a group is:

$$d = (.0041)2^m$$

where, d = mid-diameter of the group (to log. scale).

m = fineness modulus (an exponent).

It is seen that the FM is a logarithmic function of d.

The whole-number values of FM correspond to the mid-diameters of the size groups (to a log. scale); however like logarithms there are any number of values between those shown which correspond to other sizes. The FM of the sand is the summation of the fraction times the FM of that group, obtained by multiplying col. 3 by col. 4. The resulting FM is the same as that given by the more direct method of Table 1.

If we plot sieve sizes as abscissae to a log. scale (sieves equally spaced) and cumulative per cents coarser than sieves as ordinates, the FM is proportional to the area under the curve.

The FM is a logarithmic function of the size of the particle (as stated in Bulletin 1); it is not an arbitrary function as stated by Mr. Kennedy.

Uses of the Fineness Modulus

The fineness modulus as originally stated by me had three distinct functions:

(a) A measure of size and grading of separate aggregates (sand and coarse aggregate).

(b) An index to the optimum percentage in which 2 or more sizes of aggregates may be combined in a concrete mix.

(c) As a term in a formula for pre-determining the quantity of mixing water required by a given mix.

This discussion will be confined largely to usage (c).

1928 Report of McMillan and Johnson

I was at a loss to know where Mr. Kennedy gained the impression that the fineness modulus was "demonstrably unsound, and has generally been abandoned." I thought I was familiar with the literature, however, I had seen no data which establish such a conclusion. While pondering this subject I happened to turn to the 1928 report of McMillan and Johnson to which Mr. Kennedy refers; there I found the following, which had escaped my notice:

(Page 4) The data from this and the subsequent series of tests have again reaffirmed the basic character of the water-cement ratio law, and emphasized the major importance of this contribution to our knowledge. On the other hand, the data have brought out the limitations of the method of designing mixtures in which the water requirement for a given mix and consistency is determined by the fineness modulus, regardless of the other details of grading. On this latter point, it must be remembered that a great number of gradings can be represented by the same fineness modulus, and it hardly could be expected that a given fineness modulus would necessarily give the same consistency for any mix and water-cement ratio.

(Page 10) From the foregoing discussion, the data from Series 122 are seen to confirm to a very satisfactory degree the water-cement ratio law of concrete strength. They are particularly useful in emphasizing the corollary to the law—the necessity for plastic mixtures. These data are of further interest in bringing out the limitations of the method of designing concrete mixtures through the application of the fineness modulus to determine the quantity of water for a given mix and consistency.

No doubt Mr. Kennedy interpreted the above blanket statements as a demonstration of the unsoundness of the method.

TABLE 3—FINENESS MODULUS AND SURFACE FACTOR OF 33 CONCRETE SANDS

Sieve analyses given as separate per cents retained by the sieves shown.
FM = the fineness modulus of the sand.
$\Sigma\ p/d$ = surface factor computed according to method of C. T. Kennedy.
The last sand is from Mr. Kennedy's paper.
Values plotted in Fig. 1.

Sand	Separate Fractions—%							FM	Surface Factor $\Sigma\ p/d$
	Pan	100	50	30	16	8	4		
Arizona	3	5	24	39	19	8	2	2.98	405
Colorado	4	6	18	19	15	17	21	3.70	329
Illinois	0	4	40	27	19	9	1	2.91	461
Kansas	1	9	35	32	17	6	0	2.73	505
Maine	2	5	24	36	23	10	0	3.03	402
North Carolina	5	18	45	31	1	0	0	2.05	662
Tennessee	1	9	40	27	9	13	1	2.77	515
Washington	2	8	24	21	20	23	2	3.26	399
Wisconsin	4	19	27	18	15	17	0	2.72	545
Illinois	2	5	17	20	28	28	0	3.57	324
Ohio	1	1	1	19	30	48	0	4.20	239
Wisconsin	1	6	25	26	23	18	1	3.22	396
Tennessee	2	22	36	14	14	10	2	2.54	628
Oregon	0	1	4	29	53	13	0	3.75	227
Oregon	1	3	12	30	51	3	0	3.38	315
Oregon	1	5	35	17	19	23	0	3.21	417
Oregon	3	7	20	27	33	10	0	3.11	389
Pennsylvania	2	10	30	19	15	15	9	3.16	447
Pennsylvania	2	10	40	24	12	11	1	2.71	522
Texas	1	15	69	14	1	0	0	1.99	722
Morris Dam	4	9	27	23	18	14	5	3.04	431
Owyhee Dam	2	10	30	30	15	13	0	2.85	496
Hoover Dam	2	13	41	15	12	17	0	2.73	545
Seminoe Dam	2	15	34	21	13	15	0	2.73	543
Coulee Dam	6	18	22	25	16	12	1	2.67	518
Friant Dam	4	12	26	30	14	14	0	2.78	479
Tygart Dam	2	14	32	19	15	16	2	2.87	516
Conchas Dam	3	13	32	20	14	18	0	2.83	504
Wheeler Dam	2	17	44	17	9	11	0	2.47	612
New York	2	11	34	20	22	10	1	2.78	498
Pennsylvania	2	12	34	21	11	15	5	2.92	512
Pennsylvania	3	20	23	26	18	8	2	2.68	558
Kennedy	1	7	29	21	22	20	0	3.16	420

Surface Factor and Fineness Modulus of 33 Concrete Sands

Mr. Kennedy uses the area as a measure of the water requirements of a sand. Since the fineness modulus has also been used in estimating the water requirements of a concrete mix, it seems appropriate to make a further comparison of these functions.

Table 3 gives the fineness modulus and "surface factor" of 33 typical concrete sands selected more or less at random from various sections of the U. S.; the first group are the sands which headed the tables in a series of recent gradings submitted by a number of the State Highway Departments. The sands from several large concrete dams have been included. The other sands were selected with a view of covering as wide a range in fineness modulus and territory as practicable. All have been used in concrete. The values have been plotted in Fig. 1. The "surface factor" is used instead of attempting to compute areas; it is $\Sigma\ p/d$ as used by Mr. Kennedy (who omits the 0 - 100 size); it is closely proportional to surface area, although it does not take

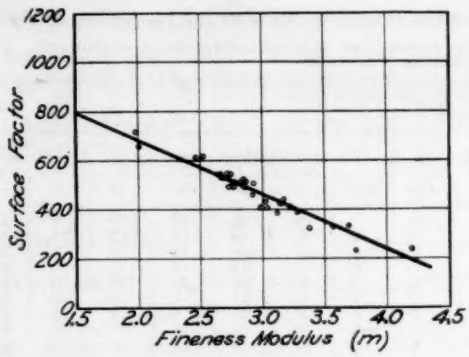

FIG. 1—RELATION BETWEEN KENNEDY'S "SURFACE FACTOR" $\Sigma\ p/d$ AND FINENESS MODULUS (m) FOR 33 SANDS IN TABLE 3

account of relatively unimportant differences in unit weights of the sands.

There is practically a linear relation between fineness modulus and surface factor. For each .01 increase in FM of the sand the surface factor is lowered 2¼ units; since the area is about 4¼ times the surface factor, we may say that each increase of .01 in FM gives (roughly) a decrease of 10 units in area. There are no doubt sand gradings which depart widely from the above figures, however the 33 sands used are probably representative of the general run of modern concrete sands.

It should be noted that when our concrete experiments of 1916-18 were under way, there were practically no specifications covering *grading of sands;* there was little processing of sands. Modern rigid grading requirements automatically eliminate many sands which were in common use 25 years ago, and eliminate many freak gradings to which attention had to be given at that time.

In general there is no necessary relation between surface area and fineness modulus of an aggregate, however for the usual run of present-day sands with "smooth grading," which I have studied thus far, there is the linear relation shown in Fig. 1.

It appears that Mr. Kennedy's surface area is, for the usual run of *modern processed sands,* a laborious method of computing a function which may with greater facility be represented by the fineness modulus.

I may be accused of being prejudiced in favor of the fineness modulus. Whatever else may be said of the FM it is a *real property* of an aggregate which can be determined almost mechanically and automatically; there is no need for short-cuts, approximations or corrections; it is easily computed and is used "as is."

Abrams' 1918 Design of Concrete Mixtures

Mr. Kennedy's criticisms of the fineness modulus were no doubt aimed at its use as a factor in the water-formula in my paper on "Design of Concrete Mixtures;" since this paper is no longer available, it will be necessary to give a brief outline of the procedure, as a basis for the following discussion.

After having carried out several thousand concrete tests using a wide range of mixtures, consistencies, size, gradings, and types of aggregates, it was found that all of the significant factors in concrete strength could be reconciled into a single general relationship by plotting compressive strength against the water-cement ratio. This gave the first water-cement-ratio-strength diagram and furnished the basis for the water-ratio law of concrete properties. This curve showed a rapid drop in strength of plastic mixes as the water-cement ratio was increased, until a water-ratio of about 1.00 (by vol.) was reached; beyond this point the decline in strength became more gradual as the curve became asymptotic to the W/C axis; for convenience W/C, which appeared as an exponent, was indicated by x.

The equation of the curve is of the form,

$$S = \frac{A}{B^x} \quad \ldots\ldots (1)$$

Where S is the compressive strength of concrete and x is the ratio of volume of water to volume of cement in the batch. A and B are constants whose values depend on the quality of the cement used, age of concrete, curing conditions, etc.

For the conditions of these tests, Equation 1 becomes,

$$S = \frac{14000}{7^x} \quad \ldots\ldots (2)$$

The relation given above holds so long as the concrete is not too dry for maximum strength and the aggregate not too coarse for a given quantity of cement; in other words, so long as we have a workable mix.

Steps in the Design of Concrete Mixes

"(1) Knowing the compressive strength required of the concrete, determine by reference to Fig. 1 the maximum water-ratio which may be used. Subsequent steps in the design of concrete mixes are only devices for securing a workable concrete using this water-ratio and a minimum quantity of cement."

The foregoing paragraphs are quoted from Bulletin 1; Fig. 1 here has reference to the 28-day water-ratio strength curve in Bulletin 1.

It should be noted that while the principles are unchanged, present-day portland cements, due to modified compound composition, finer grinding, etc., give much higher 28-day strengths than that indicated by Equation 2; in general the formula will apply to present cements if we change the 7 to about 4.

The remaining steps in the design of concrete mixes will be briefly abstracted.

(2) Make sieve analyses of fine and coarse aggregates.

(3) Compute fineness modulus of each.

(4) Determine maximum size of aggregate.

(5) Determine maximum permissible, fineness modulus of mixed aggregate for this mix, by reference to a chart based on concrete tests.

(6) Compute percentages of fine and coarse aggregate required to produce the fineness modulus desired, by using a formula,

$$p = 100\frac{A - B}{A - C} \quad \ldots\ldots (3)$$

where p is the percentage of fine aggregate, A, B and C are the fineness moduli of the coarse, mixed, fine aggregate.

The quantity of water required was computed from a "water formula" which will be discussed below. Tables and diagrams were given which enabled the user to estimate quickly the effect of changes in water-ratio, cement factor or size and grading of aggregate.

We determined that the water-cement ratio was the key to the design of concrete mixes; in order to use this knowledge as a basis of mix design it was necessary to ascertain what the principles were which governed the distribution of water in a plastic concrete mix so that a mix could be designed with a minimum W/C. It was established that:

. . . . a given water-ratio can be secured with a minimum of cement if the aggregate is graded as coarse as permissible (considering its size and the mix used) and if we use the driest mix which can be properly placed. Securing a coarse, well-graded aggregate, using rich mixes, employing the driest practicable consistency, using mechanical methods of placing concrete* etc., are all methods of producing a workable mix with a minimum water-ratio. Experience or trial is the only guide in determining the relative consistency of concrete necessary in the work. Obviously the driest workable consistency should be used.

Workability and the Water-Ratio Law

The following paragraphs bearing on workability are quoted from Bulletin 1:

(Page 2) With given concrete materials and conditions of test the quantity of mixing water used determines the strength of the concrete, so long as the mix is of a workable consistency.

(Page 3) (Referring to the water-ratio strength curve) Values from dry concretes have been omitted. If these curves were used we should obtain a series of curves dropping downward and to the left from the curve shown.

*It may be noted that this statement was made about 15 years before internal vibration of concrete became a recognized field method.

(Page 10—in italics in original). *Use the smallest quantity of mixing water that will produce a plastic or workable concrete.*

The report of McMillan and Johnson contains the following curious statement:

(Page 5) While in all of the earlier work in this connection, the necessity of having plastic and workable mixes was recognized and repeatedly emphasized, it remained for the later studies to indicate the non-workability of the mixtures as the true cause for the failure of the strength tests to conform to previous experience.

It is difficult to understand the import of this statement in view of the fact that Bulletin 1 had pointed out repeatedly that the water-ratio strength relationship did not hold unless the concrete was workable. The importance of plastic or workable concrete as a limitation on the water-ratio-strength relation, and the behavior of non-workable mixes, were clearly understood 10 years before McMillan and Johnson's report was published.

Abrams' Water Formula

Bulletin 1 gave the following formula for estimating the quantity of mixing water required to produce a workable concrete.

$$x = R\left[\frac{2}{3}\,p + \frac{.30n}{1.26^m}\right] + (a - c)n \qquad (4)$$

where x = Water required—ratio to volume of cement in batch (water-ratio).
R = Relative consistency of concrete, or "workability factor."
p = Normal consistency of cement (ratio by weight).
m = Fineness modulus of aggregate (an exponent).
n = Volumes of mixed aggregate to one of cement (the mix).
a = Absorption of aggregate, ratio of water absorbed to volume of aggregate.
c = Moisture contained in aggregate, ratio of water contained to volume of aggregate.

This formula takes account of all the factors which affect the quantity of water required in a concrete mixture. These factors may be classified as follows:

1. *"Workability" factor,* or the relative consistency of the concrete. This is dictated by the kind of work being done; concrete must be more plastic (which generally means a wetter consistency) in reinforced concrete building construction than is necessary in mass work. The term (R) in the equation takes care of this factor. (R) may vary from, say, 0.90 for a dry concrete to 2.00 or higher for very wet mixes.

2. *Cement factor,* which is made up of two parts: the *quality* of cement so far as normal consistency is concerned (p); the *quantity* of cement in the mix (n).

3. *The aggregate factor.* This includes the three terms within the parenthesis in Equation 4. The first term, involving (m), takes account of the size and grading; the second (a) the absorption, and the third (c) the water contained in the aggregate."

It was realized in 1918 that the above formula was only a first approximation. This is shown by the approximate values assigned to the constants, such as 3/2 of normal consistency of the cement. It was known that further study might require slight adjustments in some or all of these values in order to adapt the formula to all ranges

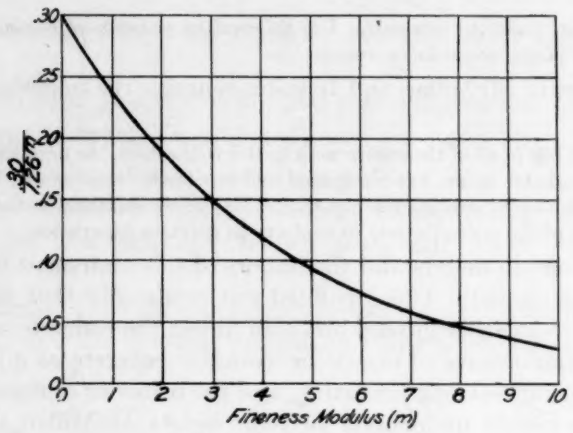

FIG. 2—VALUES FOR $\frac{.30}{1.26^m}$ IN ABRAMS' WATER FORMULA (1918).

of aggregate grading, etc. The most disturbing factor arose from an attempt to account for absorption of aggregates; if saturated surface dry aggregates were used the term $(a - c)n$ would drop out.

Our concrete tests of 1916-18 were made by considering the absorption after 3 hours (see Bulletin 1); later 30 min. was tried. It is doubtful if there is any time at which absorption can be determined which will be generally satisfactory, since particles of different size absorb at different rates; the absorption is no doubt greatly influenced by the "relative consistency" of the concrete. An absorption of 1 per cent by the aggregate accounts for enough water to change the W/C 10 to 15 per cent. The only solution for this difficulty is to make concrete studies using aggregates which are saturated with water in advance of the tests; this is now being done in many laboratories. On many large jobs; considerable sums are being expended to maintain aggregates in a uniform moisture condition until they reach the mixer.

Abrams' Water Formula Applied to Field Mixes

It will be shown that the above water formula gives surprisingly accurate results when applied to recent field mixes. In making these applications of the formula we are using no principles or data that were not contained in the original formula as published in 1918.

It is difficult to find reports in the literature which are sufficiently complete to carry out the computations for checking the water formula with the W/C actually used on the work. Table 4 gives the elements of 8 field mixes which I happened to have at hand; the time available for the preparation of this discussion has not permitted a wide search of the literature for a more comprehensive tabulation.

TABLE 4—W/C OF RECENT CONCRETE MIXES COMPUTED BY ABRAMS' WATER FORMULA

Net W/C computed by Formula (4) above, that is, water absorbed by aggregates is not included. All required elements of mix are given except normal consistency of cement which was assumed as 22% in all cases; this gives a factor .33 for the term 3/2 p in the formula.

(1) "The Effect of Vibration on the Strength and Uniformity of Pavement Concrete" by Jackson and Kellermann, Public Roads, April, 1937. In this instance we used the first example in each of the gravel and stone mixes.

(2) "Special Cements for Mass Concrete," by J. L. Savage, 1936.

(3) "Technical Report on the Construction of Morris Dam," by V. L. Peugh, Pasadena Water Department, 1935.

(4) Private correspondence from Capt. Hans Kramer, U. S. Engineer Office, Jan., 1939.

Job	Ref. No.	Aggregate			Concrete			Net W/C (vol.)	
		Kind of Coarse	Max. In.	FM Total	Type	Mix (n)	Slump In.	Used	Computed
Tests of Road Slabs.	1	Gravel	2	6.26	—	4.5	2.9	.68	.69
	1	Stone	2	5.95	—	4.5	2.7	.73	.72
Grand Coulee Dam.	2	Gravel	6	6.79	Mass	6.7	2¼	.80	.80
Hoover Dam.....	2	Gravel	9	7.06	Mass	6.7	3¼	.81	.79
Morris Dam.....	3	Stone	6	6.92	A	7.1	½	.81	.77
	3	Stone	6	6.92	B	6.3	1¼	.74	.74
Conchas Dam.....	4	Gravel	6	6.74	Mass	9.0	½–1	1.04*	.92
	4	Gravel	6	6.60	Face	6.65	1–1½	.82*	.78

*Probably not corrected for absorption; "approximately half of the gravel particles are composed of sandstone having various degrees of hardness." It is probable that excessive grinding in the mixer may account for the high W/C used.

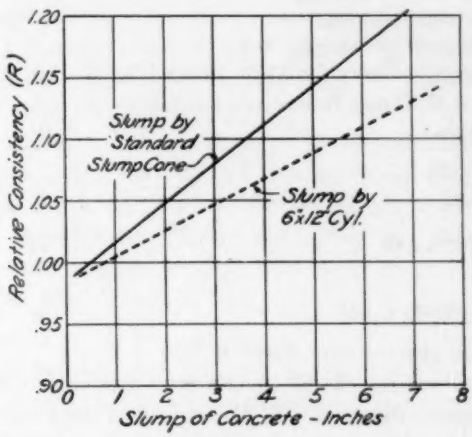

FIG. 3—RELATION BETWEEN "RELATIVE CONSISTENCY" (R) AND SLUMP OF CONCRETE

It was necessary to assume a normal consistency of the cements; this was taken as 22 per cent by weight in all cases.

It should be noted that the method of making the slump test has been changed. The slumps on which the water formula was based were determined by using a 6 x 12-in. cylinder mold; recent tests are based on the 4 x 8 x 12-in. slump cone which is now the standard. In order to evaluate the factor R in the formula, we have considered

the slump cone to give slumps 2/3 those of the cylinder. The basis of the slump values are given in Fig. 3. There was another approximation in that we assumed the slump to increase as a linear function of the W/C.

In order to simplify the computations we have considered the expression $\frac{.30}{1.26^m}$ as a single function where m (an exponent) is the fineness modulus of the aggregate; the values of this expression are plotted in Fig. 2; values for any required m may be taken from the curve.

Example of Computation of W/C

The exact steps in computing W/C are shown below, using the gravel mix at the head of Table 4. Water absorbed by the aggregates has been deducted, and water held by the aggregate has been included in the net W/C; in this way the term $(a - c)n$ of the formula does not appear.

We have the following data:

p = n. c. of cement (assumed)	= .22 (wt.)
m = FM of combined aggregate	= 6.26
n = mix (6 sacks per cu. yd.)	= 4.50
R = relative consistency for 2.9 in. slump (Fig. 3)	= 1.075

The required W/C may be computed as follows:

Item of Formula	*W/C (vol.)*
$3/2\ p = \frac{3 \times .22}{2}$	= .33
$\frac{.30n}{1.26^m} = .070^* \times 4.50$	= .315
Sum	.645
Net W/C = 1.075 x .645	= .69

Abrams' Formula Agrees with Field W/C

The computed values of .69 in the above example should be compared with .68 actually used. With the usual data concerning aggregates, mixes, etc., the computations necessary to determine W/C consist of two slide-rule multiplications.

The mixes used from *Public Roads*, April, 1937, were from single batches; the first in the gravel and stone series. Other examples are based on average practice as reported by the authorities cited.

It is interesting to note that the computed values for W/C agree with those actually used on the work, except in the case of Conchas Dam; here there is a reason to believe that the discrepancy between W/C computed and that used is due to the higher W/C required as a

*From Fig. 2, for FM 6.26.

result of excessive grinding in the mixer of sandstone gravel of "varying degrees of hardness."

Possible Adjustments of Constants in Formula

It should be reiterated that the computed W/C in Table 4 are based on information which was available when the tests reported by McMillan and Johnson were begun; information which has been available to everyone for the past 22 years. A value of 3/2 of the normal consistency of the cement may not be the best one, maybe this should be modified to, say, 1.35 or 1.65 (instead of 1.5); maybe it should be a function of the mix (cement factor); possibly the factor $.30n$ should have been $.28n$ or $.33n$; possibly the constant to which the fineness modulus (m) was attached should have been 1.25 or 1.27 instead of 1.26. Some better factor for taking care of a wide range in slump might have been devised to replace the "relative consistency" (R).

The *sole purpose* of the concrete tests which were reported in part by McMillan and Johnson in 1928 was to furnish information on the following:

(a) Proper time for absorption test of aggregates.

(b) Slump and flow as affected by changes in water content.

(c) General revamping of water formula to conform to wider range of data, based on new methods of measuring workability by means of the slump cone and flow table.

While all of those tests were outlined by me and carried out under my direction, I hasten to state that I had nothing to do with the interpretation which has been put on them. There is no evidence that McMillan and Johnson studied the tests from the standpoint of their original purpose; it is possible that they were not aware of the main objectives of the investigations.

Concluding Remarks

To be able to pre-determine the quantity of mixing water required for given materials, mixes and consistencies is the key to concrete mix design. The writer's formula enables the engineer to pre-determine the quantity of mixing water to the degree of accuracy represented by the field mixes in Table 4. This seems to be a valuable tool in the design of concrete mixes. The formula works when applied to field mixes; if laboratory results do not agree, it is possible that the fault is in the laboratory methods or in the interpretation which has been placed on laboratory tests.

To advocate "trial mixes" as a basis for concrete mixes (it is *not* design) is to admit that we have learned little or nlthing about concrete during the past century. A good water formula enables us to

make "trial mixes" in the office, probably with a greater degree of accuracy than they can be made in the field or in the average concrete laboratory.

AUTHOR'S CLOSURE

An uneasy feeling that my paper might turn out to be too long for the space available for its publication was responsible for certain sins of omission which tend to confuse the reader.

The diagram of Fig. 8 was definitely not intended for general application. The entire second part of the paper was conceived as an illustration of the actual use of the workability factor laws developed in the preceding part. Since I had confined my proof to a single series of tests,—those reported in the "Report of the Director of Research," P. C. A., 1928—it seemed fitting to use the data obtained from these same tests in demonstrating the application of the method. That is all that Fig. 8 purports to be.

In Mr. Russell's very valuable contribution—which deserves more study than I have been able to give at the moment—he relates the application of the curves of Fig. 8 to specific conditions of an actual present-day job. It is not to be expected that data derived from tests conducted at least thirteen years ago would produce quantitatively correct results to-day, due to the difference to be expected in the quality of the cement. Due to improvements in manufacture, such as finer grinding and the addition of plasticising agents at the mill, modern cements are more workable than those of thirteen years ago, and it seems reasonable to suppose that a mix designed for a 4 in. slump for the cement of the earlier period might produce a much larger slump, even as much as 9 in., with present-day materials.

Mr. Russell reminds me of another limitation to the work upon which my paper was based. The application of my method, as well as the tests upon which it was based, have been largely confined to the materials used in building construction, in which the maximum size of aggregate is limited to about 1½ in. Aggregates of the size to which he refers—6 in. maximum—are entirely outside the scope of the experimental data upon which my figures were based. It is entirely probable that adjustments must be made in the method to permit its application to mass concrete. After all, the sub-title of the first part of my paper was, "A Rational Basis for the Design of Concrete Mixes;" my contribution was not designed to end all experimental investigation but to stimulate it, to present a method of approach which has worked satisfactorily in the field for which it was designed and which may serve as a basis for further research by those better equipped for the task.

The factors which govern the range of use of a given set of workability-factor curves are (1) the quality of the cement and (2) the shape of the particles of fine aggregate. As far as the first factor is concerned, the situation is much the same with regard to the workability-factor curves as it is with regard to the water-cement ratio curve; variations occur with individual cements, but standard curves can usually be found which will apply for most of the cements available in a given market. With regard to the second factor, the same set of curves will apply throughout a wide range of gradings provided that the shape of the finer particles is substantially uniform. Curves derived from water-born sands would not be expected to apply in the case of sharp angular particles in the fine aggregate, although data are lacking to evaluate the difference. The surface area of the coarse aggregate is so small in comparison with that of the fine aggregate that the effect of the substitution of broken stone for gravel, or any other change in the shape of the coarse aggregate, is negligible.

Like the character in Moliere who was told by an expert that he had been talking "prose" all his life, I am surprised to find myself described by Mr. Peck as "definitely within the fold of the theoretical school." I can assure Mr. Peck that my efforts in the field of the design and control of concrete have been utilitarian and un-academic to a degree; my only laboratory has been the job. The situation is the more baffling since Mr. Peck, a faculty member of a Department of Civil Engineering, describes himself as a "pragmatist". Perhaps Mr. Peck is a fifth-columnist whom the pragmatists have parachuted within the cloistered precincts of the College of the City of New York.

Mr. Peck's attitude is the more discouraging to us "theorists" since there seems to be nothing we can do about it; he seems to defy us to do our worst, and counters by saying, "What's the use?" What I tried to do in my paper was to enunciate certain laws; the method of design based upon those laws is of secondary importance. The pragmatist may or may not accept my method of design, but if I am correct in my statement of the laws, he cannot avoid or deny them. Assuming the correctness of my statement of those laws, for instance, I defy Mr. Peck to go into his laboratory and mix a workable concrete, under the conditions of modern building construction, without satisfying those laws. Unless he can successfully question the validity of those laws, he must accept them, whether he wants to or not. It seems to me that a true pragmatist would not dismiss my efforts until he found out definitely that they did not work. I don't think that Mr. Peck is a true pragmatist, unless (to re-work an old gag) a pragmatist is to be defined as a man who has to live with a lot of theorists.

When I wrote that the fineness modulus method of design was "fundamentally unsound, and has generally been abandoned", I was stating what seemed to me to be obvious facts, and had no idea that I was to start a controversy. Mr. Abrams' defense of the fineness modulus therefore comes as a surprise. I know of no one who has a better right to defend the fineness modulus method than Mr. Abrams or who is better equipped for the task. Actually, the question as to whether or not the fineness modulus method of design has been generally abandoned is one that the reader will decide for himself upon the basis of his own experience, and I see no need for a continuance of the discussion on this point. It is unkind of Mr. Abrams, however, to assume that my opinion on this matter was derived solely from certain statements of McMillan and Johnson, which had hitherto escaped his attention; the interested reader, if any such there be, will find much more definite evidence elsewhere in the Report than is given in the guarded language of the excerpts which Mr. Abrams has quoted.

Mr. Abrams refers to his paper on "Design of Concrete Mixes" as being no longer available. The first sentence of my paper asserts that the history of the rational design of concrete begins with the publication of that paper, and I assure Mr. Abrams that my copy is still available. I regard it as a milestone in the history of the subject, and it is one of my prized possessions. I could wish that Mr. Abrams had read my paper with one-tenth the attention with which I have studied his; if he had, he would have avoided certain obviously unintentional mis-statements of my position.

Mr. Abrams states that "Mr. Kennedy's statement seems to imply that minimum voids in the dry rodded aggregates is the optimum condition for concrete mixes", and later refers to my "principle of minimum voids". Mr. Abrams' patience should have been extended to include the reading of Page 397 of my paper, which says "the amount of cement paste required for a given degree of workability is a minimum at a mix coarser than that giving maximum density. This is a general relationship," and Page 398, which says that "the minimum amount of water will be required at a mix coarser than that giving maximum density, and that the difference will be increased in the case of the richer mixes."

Having set up a misleading statement of my position, Mr. Abrams demolishes it by the following quotation from his Bulletin 1: "The aggregate grading which produces the strongest concrete is not that giving the maximum density (lowest voids). A grading coarser than that giving maximum density is necessary for highest concrete strength."

Mr. Abrams is obviously unaware of the fact that I used the very same quotation, on Page 398, to demonstrate the validity of my own analysis.

Since we both prefer a mix coarser than that giving minimum voids, there can be no difference between Mr. Abrams and myself on this point; in fact, I hold that my analysis explains on rational grounds a phenomenon which Mr. Abrams discovered by purely empirical methods. (Perhaps Mr. Peck is right; perhaps I am a theorist, after all.)

Passing from voids to surface areas, Mr. Abrams says, "In the example chosen by Mr. Kennedy the neglect of the area of the 0-100 sand made little difference, since there was only 1 per cent of this size". Here, again, Mr. Abrams must have missed several pages of my paper. I refer him to Pages 384-387, where my analysis was applied to mortar mixes with seven different gradings of sand, with a percentage passing the 100 sieve varying from 0 to 5 per cent. Furthermore, there was no coarse aggregate, so that in the case of mortar A, for example, 5 per cent of the entire aggregate was less than the No. 100 sieve size.

Mr. Abrams seems to chide me, by inference, for neglecting the 0-100 sand in computing the surface area. Why shouldn't it be neglected? The cement paste consists of discrete particles surrounded by water; it is impossible to conceive of this paste forming a film around particles which are of the same order of magnitude as the particles of cement themselves. Much of the material passing the No. 100 sieve is not much larger than the larger particles of the cement. These small particles act as void fillers but can hardly be held to require a surface film for their lubrication in the mass. Instead of blandly assuming that the surface area of the 0-100 sand must be included, it is up to Mr. Abrams to give some logical reason why it should not be neglected.

I hold that the burden of proof is on Mr. Abrams and that the facts are against him. Consider the results of my analysis as applied to the seven sands described in the table at the head of Page 385 of my paper, where the amounts passing the No. 100 sieve varied from 0 to 5 per cent of the entire sample. This is certainly as wide a range as will be ordinarily encountered in practice. If any important error is introduced by the neglect of the 0-100 sand, the effect should be evident here. Yet the workability curves for the range of mixes given in Table 2, as shown by Fig. 5, exhibit a really remarkable degree of uniformity. The neglect of the 0-100 sand is justified in the event. (This, Mr. Peck, is true pragmatism).

On the whole question of the surface area of the aggregate, I definitely belong with Mr. Peck and his fellow pragmatists. Mr. Abrams criticizes a lack of preciseness in my evaluation of the surface area. The fact of a lack of precision is not a valid criticism unless it can be shown that precision is necessary or desirable. On Page 382, I wrote: "The results thus obtained should be a fairly close approximation in the case of water-borne particles of typical sand deposits; it should be noted that exact values are not necessary provided that the values used are in some fairly constant ratio to the actual areas." In the aggregates of Diagrams A and B on Page 393, the amount of cement paste required to provide the necessary thickness of film around the individual particles amounts to one-half to one-third the amount required to fill the voids in the aggregates, or one-third to one-fourth of the total amount of paste required for the given degree of workability. Consequently, errors in the determination of the surface area are permissible to the extent of three or four times the permissible errors in the determination of the voids in the aggregate, and a degree of approximation is therefore justified.

As against Mr. Abrams, I claim the highest authority in the determination of surface areas. On Page 9 of Mr. Abrams' Bulletin 1, "Design of Concrete Mixes," he gives the surface areas of a number of different gradings of aggregates. He gives no information as to the method used in computing these areas, but careful analysis shows that he has included the 0-100 on the assumption that the material passing the No. 100 sieve bears the same relation to the material retained on the No. 100 sieve as the material retained on any sieve bears to that retained on the next larger sieve; that is, that the 0-100 material has twice the surface area per pound as the material retained on the No. 100 sieve. Making the same assumption, and applying the crude and bungling methods which I use in my paper, I get substantially the same surface areas as are given by Mr. Abrams, my results being substantially proportionate to his but about 5 per cent higher. With the exception of the inclusion of the 0-100 material, my methods then are substantially equivalent to those actually used for publication by Mr. Abrams, in an attack upon another method of design. Whatever others may say about my methods of computing surface areas, Mr. Abrams must, to be consistent, limit himself to the single item of the inclusion of the 0-100 material.

Mr. Abrams' claim to fame is too well established to be affected by this discussion. The importance of the subject of the paper, if not of the author, justifies a less superficial treatment than he has given it.

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Proposed Specifications and Methods of Testing for Concrete Staves to be Used in Farm Silo Construction*

A Progress Report of Committee 714, Recommended Practice for the Construction of Concrete Silos

C. A. HUGHES†, Chairman

In 1926 the Institute adopted "Standard Specifications and Building Regulations for Concrete Staves," Serial Designation P-4A-26[1]. No amendments have been made to this specification in the subsequent thirteen years.

After thorough discussion of existing data, Committee 714 recommends withdrawal of this specification and the adoption of "Specifications and Methods of Testing for Concrete Staves to be Used in Farm Silo Construction" presented herewith in Appendix 1 for the consideration and action of the Institute.

For the past two years the committee has been and still is engaged in gathering data which will permit writing a "Recommended Practice" which will replace the sections of specifications P-4A-26 dealing with building regulations.

For the information of Institute members interested in this subject a summary of a subcommittee report on "Methods of Manufacture, Machines, and Types of Staves" is given in Appendix 2.

This report (including Appendix 1 and 2) was submitted to letter ballot of the Committee which consists of 13 members; 13 members returned their ballots of whom 12 have voted affirmatively and 1 negatively.

*Received by the Institute Nov. 29, 1939. Released by the Standards Committee for publication as information and for discussion.

†Associate professor of structural engineering, Engineering Experiment Station, University of Minnesota.

[1]*Proceedings* Amer. Concrete Inst., Vol. 22, 1926, p. 666.

APPENDIX 1—PROPOSED SPECIFICATIONS AND METHODS OF TESTING FOR CONCRETE STAVES TO BE USED IN FARM SILO CONSTRUCTION

Specifications

1. *Scope*—These specifications describe physical characteristics to be used as the basis for acceptance of unreinforced concrete silo staves for use in farm silos made from a mixture of Portland cement and sand, gravel, or crushed stone aggregate.

2. *Materials*—The cement shall satisfy the requirements of the current A.S.T.M. standard specifications for portland cement or for high-early strength portland cement. The aggregate shall comply with the current A.S.T.M. tentative or standard specifications for concrete aggregates.

3. *Flexural Strength*—(*a*) Flexural strength of staves with plane faces. When calculated as in paragraph 12 f, the average flexural strength of five staves at the time of delivery shall be not less than 120 lb. per in. of width, and the flexural strength of an individual stave shall be not less than 115 lb. per in. of width.

(*b*) Flexural strength of staves with non-planar faces. When calculated as in paragraph 12 g, the average flexural strength of five staves at the time of delivery shall be not less than 690 p.s.i. and the strength of an individual stave shall not be less than 660 p.s.i.

Note: The flexural stresses of 690 and 660 p.s.i. correspond respectively to flexural strengths of 120 and 115 lb. per in. of width for plane-faced staves when calculated in accordance with section 12 f.

4. *Absorption*—The absorption after submersion in water for 24 hours shall not exceed 6 per cent by weight.

5. *Minimum Thickness*—No stave shall have a thickness at any section less than 2.0 in.

6. *Marking*—All silos shall bear a distinctive mark of the manufacturer of the staves used in its construction.

Inspection and Rejection

7. *Inspection*—Proper facilities shall be provided the purchaser for sampling and inspection either at the factory or at the site of work, which ever may be specified in the contract. At least ten days from the time of sampling shall be allowed for the completion of the tests. All tests shall be made in accordance with the methods specified in sections 12 and 13.

8. *Expense of Tests*—The expense of inspection and testing shall be borne by the purchaser, unless otherwise specified in the contract.

Methods of Testing

9. *Selection of Staves for Test*—Staves for testing shall be selected by the purchaser or by a competent representative authorized by him to do this work. Such staves shall be representative of the lot of staves from which they are selected. Full-size staves shall be used in all cases.

10 *Number of Staves per Sample*—The sample for testing shall consist of five full-size staves.

11. *Marking*—All staves selected for testing shall be permanently identified before shipment to the laboratory.

Flexural Tests

12. *Moisture Content at Time of Test*—(*a*) The staves shall be submerged in water for 24 hours prior to testing. They shall be loaded immediately after removal from submersion. Surface water may be wiped off with a damp cloth.

(*b*) *Measure of Thickness*—The thickness of plane-faced staves shall be taken as the average of five measurements made with a thickness gage reading to 1/1000 in. These measurements shall be taken along the short axis (or at the break) of the stave at one inch from each edge, at the quarter points, and at the center point. This average shall be recorded to the nearest 1/100 in.

(*c*) *Width*—The width of plane-faced staves to be used in calculating the flexural strength shall be the laid-up width. This width shall be taken as equal to one-fourth the distance measured between corresponding points on the first and fifth staves of five staves fitted together on a plane surface, so as to obtain the minimum overall width.

(*d*) *Position*—The stave shall be supported on a 24-in. span and loaded at midspan. The surface of the stave intended to be on the exterior of the silo shall be in tension under the test load. The end supports shall be free to rotate in directions parallel and perpendicular to the long axis of the stave and shall be sufficiently stiff so that the load is substantially uniformly distributed along their length. Padded bearing plates 2 in. wide shall be used at the supports and at the central knife-edge. The set-up shall be such that no torsional moments are developed. Special bearing blocks shall be used when testing non-planar staves.

Note: Fig. 1 shows the details of an apparatus for testing silo staves, which complies with this specification.

(*e*) *Speed of Loading*—The speed of the moving head of the testing machine shall not be more than 0.05 in. per minute.

(*f*) *Flexural Strength*—The flexural strength of plane-faced staves shall be determined by the following calculation:

$$\text{Flexural strength} = \frac{\text{maximum load in pounds}}{\text{width in inches}} \times \left[\frac{2.50}{\text{average thickness in inches}}\right]^2$$

Note: The quantity in brackets has the following values for the stave thickness shown.

Stave Thickness	Multiplication Factor
2.0	1.56
2.25	1.23
2.5	1.00
2.75	0.82
3.00	0.69

Note: Fig. 2 will be found convenient in correcting flexural data for staves having thicknesses other than those listed above.

(*g*)—The flexural strength of staves with non-planar faces shall be determined by the following formula:

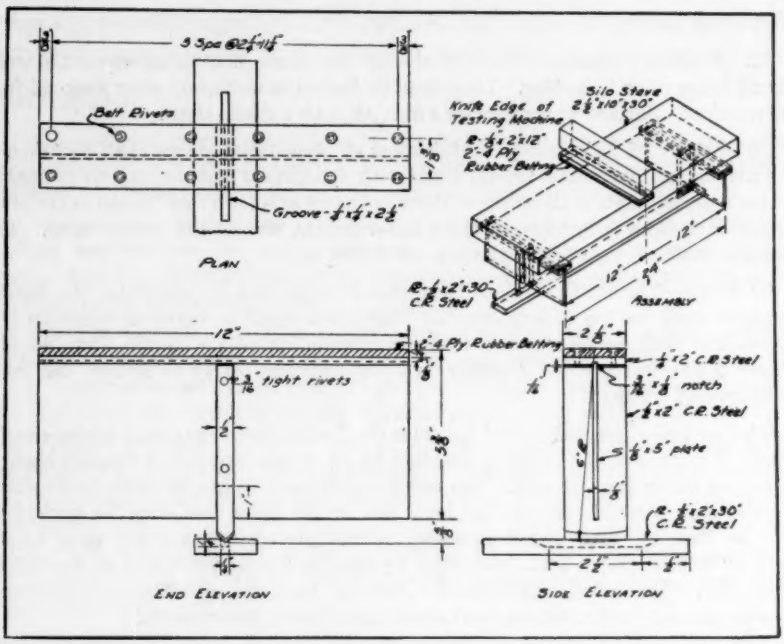

FIG. 1

$$\text{Flexural strength in p.s.i.} = \frac{6\,W\,c}{I}$$

in which, W = the maximum load in pounds
c = the distance from neutral axis to extreme fibre in inches
I = the moment of inertia of a section normal to the long axis of the stave in inches to the fourth power.

Absorption Tests

13 (*a*) *Selection of Test Samples*—The sample shall consist of one of the two pieces approximately a half-stave) of each of the five staves which result from the flexure test.

(*b*) *Marking*—Each piece shall be marked so that it may be identified at any time with the stave from which it was taken. The marking shall not cover more than 5 per cent of the area of one face of the half-stave.

(*c*) *Drying*—The pieces shall be dried in an oven at a temperature between 100° and 110° C. (212° and 230° F.) and weighed at 24-hour intervals until the loss in weight does not exceed 0.2 per cent of the previous weight.

(*d*) *Accuracy of Balance*—The balance used shall be sensitive to within 0.05 per cent of the weight of the smallest piece tested.

(*e*) *Immersion*—The dry weights of the pieces shall be obtained after which they shall be immersed in water at room temperature (60° to 80° F), for 24 hours.

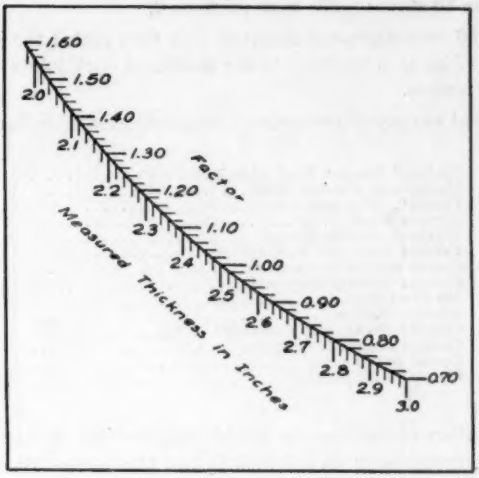

FIG. 2

They shall be removed from the water and allowed to drain for 60 seconds by placing on ⅜ in. or coarser wire mesh, visible surface water being removed with a damp cloth, and immediately weighed.

(*f*) *Per cent Absorption*—The absorption is the difference between the wet and dry weight of the sample divided by the dry weight and multiplied by 100.

APPENDIX 2—REPORT OF SUBCOMMITTEE OF COMMITTEE 714 ON METHODS OF MANUFACTURE, MACHINES AND TYPES OF STAVES

Your Subcommittee sent a questionnaire to all manufacturers of concrete stave silos in the United States. In addition, H. B. Emerson, the other member of this Subcommittee brought this questionnaire personally to the attention of a large number of concrete stave manufacturers through his service engineers. As a result, data have been received from 49 concrete stave manufacturers. One manufacturer made two types of staves.

A review of these data shows that of the 50 machines for making staves, 35 were of the power tamp type; 11 vibration type; 3 were of the hand tamp type, and 1 used hydraulic pressure.

The majority of staves made today are 30 in. long, 10 in. wide and 2½ in. thick. The following table is an analysis of the length, width and thickness of staves as made in 50 plants.

STAVE DIMENSIONS

Length Inches	No. of Plants	Width Inches	No. of Plants	Thickness Inches	No. of Plants
24	1	10	37	2	6
28	7	11	1	2½	44
30	41	12	12		
36	1	12½	2		
	50		52		50

Four only of the 50 staves made were reinforced.

Sixteen of the 50 manufacturers reported that they sealed the joints with mortar in erecting silos. This is in addition to the sealing of such joints through the application of interior washes.

There was a great variety of protective treatments used as indicated by the following summary:

Portland Cement Wash plus Anti-Hydro	1
Cumar over Cement Wash	1
Cement, Alum and Lye	6
Cement Wash	19
Portland Cement Plaster	9
Cement Wash and Iron Filings	1
Cumar and Chinawood Oil	1
Cement Glazing Preparation	1
No Treatment	1
Vitrified Facing	1
Cement Plaster and Resisting Coating	1
Cement Plaster Facing plus Wax	2
Cumar over Plaster	1
Cumar or Linseed Oil	2
	47

This wide variation of treatments would indicate that the industry is doing a great deal of experimenting in an attempt to find the best solution of this problem.

Respectfully submitted,

(Signed) W. G. KAISER
H. B. EMERSON

Discussion of this report should reach the Institute Secretary by April 1, 1940—for committee consideration and publication in the* Supplement *to be issued with the* JOURNAL *for September, 1940.

Vol. 36 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE
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Discussion of a Progress Report of Committee 714, C. A. Hughes, Chairman

Proposed Specifications and Methods of Testing for Concrete Staves to be Used in Farm Silo Construction*

BY C. A. HUGHES AND P. M. WOODWORTH

CONVENTION DISCUSSION

Professor Hughes—This report was released by the Standards Committee for publication and discussion only—not for immediate adoption. Those interested probably read it, and this is your opportunity for discussion.

P. M. Woodworth†—In reviewing the specification, two things occur to me: first, I believe the absorption rate should be considered in specifications as a quality feature that will determine, to a considerable extent the durability under acid exposure. The second is a minor matter: As a stave is tested in a saturated condition, the further procedure in determining absorption is to dry it and then determine the absorption. It seems to me it facilitates the testing of the staves as far as absorption rate is concerned, to put it back in water for an hour or two and obtain its gross weight and then dry it out. In this way I think we will get a true indication of the absorption by saturating it first and drying it out second and decrease the time required for determining the absorption rate.

Professor Hughes—Yesterday afternoon and again today the Committee is discussing a revision of the proposed absorption specification. We propose to run a series of cooperative tests which may permit some time in the future the inclusion of a durability test. It is hard

*JOURNAL, Amer. Concrete Inst., Feb. 1940.
†Portland Cement Association, Chicago.

to get agreement on a durability test, whether it is for silo staves or anything else, and it will take us a little time to arrive at something satisfactory to the research men and to the manufacturers of silo staves.

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *FEBRUARY 1940*

Job Problems and Practice

Some will ask questions — Some will answer them — Some will do both

A. C. I. members are invited to use this new JPP department as an informal means toward mutual help. When a problem holds possibilities of general interest the discussion will be briefed in these pages.

If you have a problem—present it; a question, ask it. If you know the answer to a question asked, or if you can contribute something which may help in the solution, or have reason to disagree with the answers or suggestions published, your contribution will be welcome.

If you know of an interesting problem whose solution has already smoothed the way of someone in the field, tell us about it—some other A. C. I. member may need just that information.

The "answers" are the answers of individuals to whom the questions are referred and not of the A. C. I. as an organization.—EDITOR

How to keep horizontal construction joints moist while carpenters are at work? (36-56)

Q—How can the horizontal construction joints in a mass concrete dam be kept moist while cluttered up with carpenters and their material for erecting special formwork (sluice tunnels, staircases etc.)? We have tried lengths of 1 in. G.I. pipe joined with rubber hose and fitted with irrigation "Skinner Nipples" at 18 in. centers, facing alternately left and right. This is excellent at week ends but cannot be used when men are working nearby. Building paper would not hug the concrete surface and was easily damaged. Our most promising idea to date is the use of brattice cloth (a heavy burlap) mats, 16 x 14 ft. which can be hosed down each morning before the carpenters start working.*

*This question and two others come off the job—a dam—from an A. C. I. member in Australia.

A—The problem of curing concrete is simple in the specification stage, but at times becomes so complicated on the job that there is a strong tendency to slight the whole matter. Usual practice, I'm afraid, is not so conscientious as this inquirer is in his requirements. Too often the sprinklers are placed in operation and then shut off to permit the carpenters to complete their work. Where exposures are for extended periods sand blankets have been used to advantage, though with this method there are often areas which are scraped dry. If clean sand is used, an excellent surface is found when the sand is washed off. The Bureau of Reclamation has used this kind of protection.

It would be interesting to know whether the mats mentioned in the inquiry produced appreciable staining of the concrete surface. —I. L. TYLER*

This is a matter which always presents some difficulties and frequently causes considerable bickering and nagging between engineer and contractor. The suggestion of covering with pads of burlap cloth wetted at frequent intervals should serve the purpose, but would require that the cloth be rolled back from the edges during form work and it would be a nuisance to place, remove and care for the large number of pads. I can visualize about as much difficulty as with the method used on Ross Dam, where 2 in. pipe for both air and water under about 100 lb. pressure are brought up with the concrete on the down stream face of each section. Manifold heads

*Pennsylvania Turnpike Commission, Harrisburg, Penna.

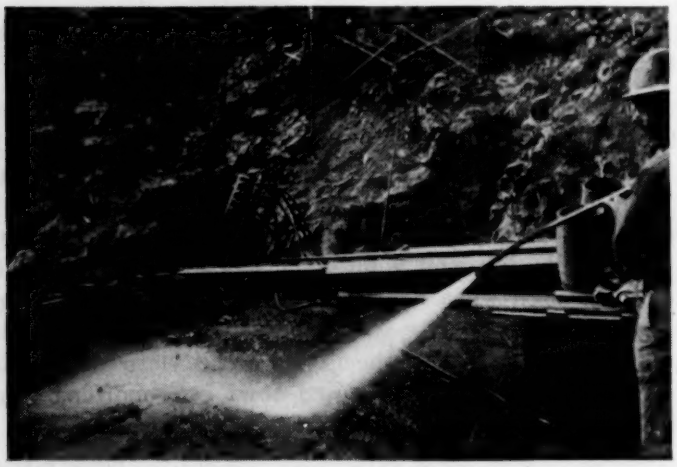

FIG. 1

are attached to these pipes (see Fig. 1) permitting easy connection of several lines of hose for both air and water to provide for cutting and washing of surfaces as well as providing easy attachment whereby hose lines with sprinkler heads at fairly close intervals may be laid and operated over the whole area while men are not working or may operate on any portion not occupied by workmen.

Men are assigned to keep the surface wet by hand spraying while men are working on the section. This causes some complaint by carpenters, surveyors and others but they soon realize that the requirement of keeping the surface wet must be enforced, even if they don't realize its importance and manage to do their work on wet surfaces by wearing waterproof foot wear and protecting themselves as best they can.

This method may not be as convenient as others but it is effective and is the contractor's method of complying with the usual requirement that horizontal surfaces shall be kept continuously wet until covered by fresh concrete.—H. F. FAULKNER.*

The best answer to this question is believed to be about one inch of wet clean sand, the same as used in the concrete. At convenient daily intervals not conflicting with the formwork operations, the coverage and moisture should be checked and maintained by raking and sprinkling.

This method of curing has been used on several dams in this country primarily because of the protection it affords to the initial air and water jet cleanup. Where the sand is not used, and the curing is by sprinkling, the fresh, bright initial cleanup usually becomes so coated and contaminated before it is time for the final cleanup that little short of sand blasting is capable of removing the coating completely. When the surface is cured and the initial cleanup is properly protected by the wet sand, the condition of the surface prior to the final cleanup is such that little if anything more than washing with the air and water jet is necessary for an excellent bright cleanup for concreting.

Principal objection to this method is the work required to obtain thorough removal of the sand and the necessity of handling the quantities of it required to and from each lift. Since sandblasting has been developed to a highly effective and economical procedure, and since it has been demonstrated that hand or mechanical sprinkling with occasional short periods of dryness is adequate curing for joints in mass concrete, the present trend is toward elimination of initial

*Engineer of Concrete Control, Ross Dam.

cleanup and removal of the surface film and contamination by sandblasting as a final operation ahead only of the washout for the new concrete.

This digresses from the specific question, but the subject of joint curing is very intimately tied up with the matter of joint clean up. —L. H. TUTHILL.*

What equipment to lift face forms of dam? (36-57)

Q—What easily transported gear is available to lift side and face forms on the monolith (26 ft., 37 ft. 4 in., and 52 ft. wide) of a mass concrete dam? The heaviest forms, the side contraction joint forms are 10 ft. wide, for a 6 ft. lift and weigh approximately 1.7 (U. S.) tons. All forms are of steel. At present a 10-ton cableway is used for form lifting, but it is desired to keep this occupied as much as possible removing excavation and placing concrete.†

A—To avoid tying up concrete placing equipment in order to raise panel forms on dams, engineers have generally turned to the use of manually operated portable hoists. At Grand Coulee dam, for raising 50 ft. by 5 ft. wood panels, the contractor used three welded pipe "A" frames tied back to concrete form anchors with light cables and chain jacks and with a second chain jack used on each frame to lift the form. This permits both horizontal and vertical movements of the form panel.

At Seminoe dam, wooden tripods with easily disconnected bolted joints and cable tie backs were used in the same manner on similar forms. The hoists were two-ton Coffing jacks. No jack was used on the tie back. The actual raising of a form required two minutes.

At Copper Basin dam three 12-ft. high "A" frames made from welded 1½ in. pipe were used to hoist a 5 ft. by 50 ft. face form. The "A" frames had metal plates to spread the load on the green concrete and were tied back to ⅜ in. round form anchors with manila rope.

The hoist was a chain link guy wire tightener similar in operation to a Coffing jack. The forms were constructed with two laminated walers each made from four 1 x 12 boards with 2 x 6 studs, 1 x 10 shiplap backing, and ¼ in. plywood facing.

At Flathead dam the forms were heavy 10 ft. by 12 ft. panels made with 10 x 10 walers, 4 x 6 studs, and 2 in. shiplap lagging. Ordinarily these were handled by the whirley crane which placed the concrete but in cases where the crane was too busy, form panels

*Bureau of Reclamation, Denver.

†This is the third of a series of three interesting questions from an A. C. I. member in Australia One (36-55) was published in January with discussions.

were hoisted by an "A" frame about 16 ft. high made with 2 x 6 lumber. The hoist was a geared hand winch mounted on the "A" frame. The "A" frame could be moved within the block by the form-setting crew but required the crane to move it from one block to another.

At Norris dam where the blocks of concrete were large, an "A" frame mounted on a small pneumatic-tired tractor was used to raise and set panels. The tractor was moved from one block to another by the concrete placing cableway, but moved within the block under its own power.—R. W. SPENCER.*

Compressed air jets as a means of placing concrete (36-54)*

A—In order to obtain some basis for comment other than an unfavorable and skeptical first reaction, we made a few exploratory trials of the method to consolidate concrete by the use of compressed air released from a tube inserted in the fresh concrete. Our first procedure was to attempt to duplicate the results claimed in Mr. Davis' letter—that of settling a 24-in. depth of concrete three-fourths of an inch by means of compressed air after the concrete had been thoroughly spaded.

After about five minutes of air application from a supply at 60 p. s. i., the surface (which had bubbled like the mud pots in Yellowstone) was smoothed off and, when overflowed concrete had been restored, was found to be at the same elevation as before the air was turned on. The form was then inclined and the side panel removed so that the concrete could be examined. Coarse aggregate had settled so that none remained in the top three inches or so. Concrete in the lower portion had settled quite tightly, had lost its plasticity, and seemed somewhat drier than it was originally. Also it appeared likely that the rising air had created many small channels which failed to collapse completely when the air was turned off. The formation of these channels was more clearly visible when we placed a batch of concrete in a sloping form with a transparent face through which we could see the action of the air, and, subsequently, of free water traveling down the channels. When the air was used without preliminary hand tamping, some rock pockets were not eliminated.

At the completion of the visual tests, eight 6- by 12-in. cylinders were cast. Four were hand-rodded and four were worked with compressed air released near the bottom of the mold from a pipe inserted from the top. The hand-rodded specimens showed a slightly higher

*See JOURNAL Amer. Concrete Inst., Jan. 1940, p. 317.

strength at seven days, with an average of 2,570 p. s. i. as compared to 2,400 for the air-worked specimens. The average weight per cubic foot of the concrete in the hand-rodded specimens (150.2 lb.) was also higher than that for the air-treated specimens (147.4 lb.).

From these results it is our conclusion that the use of air as suggested is incapable of properly consolidating the concrete without preliminary hand work and that it is likely to be detrimental in causing stratification and "piping." The air would be used to better advantage through a good air vibrator. The higher strengths and densities reported by Mr. Davis are not understood, but may have come about through reduction in water content as the air passed through the concrete, and loss of mortar and water from the tops of the test specimens. The reported settlement in the 24-in. depth may have been due to very poor preliminary hand working or to expelling surface concrete from the forms, or both.

A practical lesson may be drawn from our test observations. It has to do with the manner in which compressed air is used to assist the concrete pump in placing tunnel arch concrete and filling overbreak. Some operators let a comparatively small stream of air flow continuously with the concrete and open the valve from time to time to clear the end of the pipe, to make the pumping easier, or to prevent plugging. This occasional blast through the fresh concrete probably does little harm if the pipe outlet is kept buried, and is known to be of considerable assistance in the placing operation; but the continuous flow of air is capable of duplicating in a measure the objectionable effects noted in our laboratory experiments.

That caution must be exercised in the use of air to supplement the concrete pump was recently demonstrated on a tunnel-lining job where considerable trouble was experienced from sticking of the oiled metal forms on removal, with resultant pitting of the concrete surface. According to reports, the trouble was finally traced to splattering of the forms with mortar blown from the concrete, and was eliminated by confining the injection of air to times when the concrete-pipe outlet was well buried.—ARTHUR RUETTGERS*

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institution's Reviews

Protective measures against foundation failures in mining (surface) structures

DR. LEUTKENS, *Beton und Eisen*, Vol. 38 No. 18, p. 289 (Sept. 20, 1939) Reviewed by A. U. THEUER

Complete and partial measures adopted to prevent foundation failures in types of structures peculiar to mining operations, coking (oven) structure, sulphur refineries, coal towers, etc., are described.

Plain impregnated concrete pipe

DR. ING. ROS, Report No. 123, Eidg. Materialprüfungsanstalt, Zurich 1939.
Reviewed by A. U. THEUER

This publication contains a complete report of the laboratory and field tests made on a commercial concrete pipe manufactured under a special process to determine its physical properties and resistance to acid groundwater attack.

Appended to the paper is a soil map of Switzerland as well as the Standard Swiss Specifications for Concrete Pipes.

Modern concrete technology

ERICH HONIGMANN, *Beton und Eisen*, Vols. 17, 18 and 19 (p. 281) Sept. 5, 1940.
Reviewed by A. U. THEUER

These three articles provide a good summary of the present state of knowledge concerning the raw materials, (cement, aggregates, water) of concrete, its manufacture, placement, and properties as controlled by routine tests or exhibited in extensive research programs. A good bibliography covering mostly sources from German-speaking countries is included.

Contribution to the study of slag cements

R. FERET, *La Revue des Materiaux de Construction et de Travaux Publics*, Nos. 352 to 359, inclusive, 1939. Reviewed by P. H. BATES

Running through nine issues of this journal Feret presents a series of articles on slag cements. This voluminous contribution covers the subject in Feret's usual exhaustive manner. The articles cover the history of these cements; the nature of suitable slags and how to determine their suitability; the properties of slag and portland cement or lime used in the blends forming these types of cements; the strengths of the various cements in flexure and compression; activators for slag; various chemical tests for durability and resistance to sulphate waters, alkalies, etc. This is a somewhat lengthy but very valuable contribution to our knowledge of slag cements.

Reinforced concrete towers for passenger car cable-ways

R. FRANK, *Beton und Eisen*, Vol. 38 No. 19, p. 297 (Oct. 5, 1939) Reviewed by A. U. THEUER

The author describes in considerable detail the characteristics of the supporting towers of a large number of cable ways intended for passenger service. On the basis of his study the following conclusions are advanced. The first cost of reinforced concrete towers is generally higher than that for comparable steel towers, although in certain very special cases the reverse may be true. The differences in the cost of transporting different materials to the building site is of minor importance since auxiliary cableways are generally used in construction. This type of structure is generally assumed to have a life from 40 to 60 years and hence classified as a permanent structure. Cost of upkeep for concrete towers is considerably less than for other types, particularly so if located near salt water.

The most advantageous statical shape is considered to be a hollow cone-like structure with a round or octagonal cross-section. The shape of the tower may be considered to be independent of the clearance provided by the beam-shoe supports since these members can be given considerable overhang.

Action of fresh and sea water on cements, limes, and concretes

E. MAYNARD, *La Revue des Materiaux de Construction et de Travaux Publics*, Nos. 355 to 359, inclusive, 1939. Reviewed by P. H. BATES

The author, a retired engineer of Ponts et Chaussees and formerly in charge of the Lime and Cement Laboratory at Rochelle, has always held a view quite different from that held by all others regarding the disintegration of cement in fresh and sea water. He holds that since 1880 it has been known that the true and principal cause of disintegration is due to the solution of lime ot the cements and its "exosmosis" into the surrounding water. After citing the principal theories on this subject and calling attention to their inadequacies, he presents arguments and data to support his viewpoint. These data are based upon specimens exposed at Rochelle, some of which have been under observation since 1852. Those who have accepted the almost classic theory that decomposition of cements in sulphate waters is due to the formation of sulpho-aluminate of lime will be interested in the very cogent arguments and data showing that this formation has a very minimum effect on disintegration.

The author reviews in the 360th issue of this journal further discussions on the subject, expounding especially on the various theories covering the action of water on concrete.

A mechanical spreader for pavement concrete

Concrete Highways and Public Improvements, Vol. XX, No. . 5, Sept.-Oct. 1939.
HIGHWAY RESEARCH ABSTRACTS

During the past summer the Pennsylvania Division of Highways built some test sections of portland cement concrete highways using a new piece of equipment, a screw concrete spreader which through the use of a cast steel screw such as found on a screw conveyor, substitutes machine spreading for hand spreading of the concrete. The concrete is dumped on the subgrade in the usual manner. The screw spreader moves through the pile, distributing it laterally, first in one direction and then in the other, so that the final result is uniform distribution of the concrete throughout the slab. At the same time additional mixing of the concrete is provided. The screw may be raised and lowered, and runs along the forms in the same manner as a finishing machine.

Test slabs were cut from the pavement after 14 days of curing and cured an additional 14 days in the laboratory yard under cotton mats. These 2 ft. by 6 ft. by 7 in.

slabs were tested in cross bending with third-point loading at 28 days. The average break for seven sets of 5 slabs each was 932 p. s. i. The modulus of rupture obtained for the 6 x 8 x 48 in. beams molded on the job, representative of the test slabs, was 891 p.s.i.

Influencing the strength of alumina cements at early ages

KEIL and GILLE, *Betonstein Zeitung*, Oct. 10, 1939, p. 261. Reviewed by A. U. THEUER

The following conclusions are reached from a study of factors affecting the time of set of alumina cements: (1) Alumina cements, unlike portland cements, cannot be mixed for prolonged periods. The maximum period of mixing was found to be 40 minutes for a 1:6.2 concrete having a W/C ratio of 0.6. It is essential that cooling be prevented during the time of mixing.

(2) Increasing the time of mixing from 2½ to 15 minutes or over results in a considerable increase in early strength of concretes regardless of the temperature at which the concrete is subsequently cooled. An increase in strength is still noticeable 16 to 18 hours after time of set.

(3) Additions of lithium chloride increase the strength of alumina cements at all temperatures. The degree of effectiveness, however, varies with the type of cement. A careful study to determine the proper percentage of lithium chloride is required for each individual cement, as too small an amount is not effective and too large an amount results in flash-set, and the range in between is very small.

(4) If high strength within severe limitations of time is required on an actual job, an increase in time of mixing is recommended in preference to the use of chemical accelerators.

Box girder bridge at Yakima, Washington

O. E. BRASHEARS, *Civil Engineering*, Vol. 9, No. 10, Oct. 1939, pp. 587-589. Reviewed by J. R. SHANK

A concrete box girder of three longitudinal cells was constructed continuous over 5 supports, the interior spans being 130 ft. and the end cantilever spans 40 ft. The box girder was 21 ft. wide and from 6 to 8 ft. deep, 6 ft. for the central 74 ft. and haunched down in 24 ft. to 8 ft. at the supports. The top slab, 7¼ in. thick over the central cell and 6¼ in. thick over the side cells, was extended 5 ft. on both sides to carry sidewalks 3 ft. 6 in. wide and railings. The distance between curbs was 21 ft. The four webs were 8 in. thick and the bottom slab 6 in. thick.

The supporting piers were set on a skew of 20 degrees. At all but the central pier the box girder was carried on concrete filled steel pipe rollers between steel plates. At each of these piers, diaphragms and three pipe rollers were placed in echelon. Intermediate diaphragms 6 in. thick were placed across the cells in the same formation, 5 sets for each 130 ft. span and one set for each end span.

The tension steel was spread, uniform spacing across the entire bottom slab. One-half in. round U bars, spaced 12 in. were placed under the longitudinal steel and 2 ft. up into the webs to act, with the web stirrups to tie the bottom slab into the webs.

A cableway and buggies served to transport the concrete. The bottom slab bottom form made an excellent platform for handling. Top forms were used for the bottom slab only near the webs. The bottom forms of the top slab were left in.

Upon removal of false work the end spans deflected ⅞ in. and the interior spans ⅜ in. The average cost for the concrete with forms was 27 dollars per cu. yd.

Le Ciment metallurgique sursulfate

L. BLONDIAU, *La Revue des Materiaux de Construction et de Travaux Publics*. Starting in No. 350 and not finished in No. 360, 1939. Reviewed by P. H. BATES

This type of cement is not made in the United States. It is prepared by grinding quickly cooled basic blast furnace slag with small amounts of portland cement

clinker or quicklime and such an amount of sulphates that the sulphuric anhydride exceeds 5 per cent. In practice the sulphate insisted upon is the mineral anhydride (not gypsum).

This series of articles started in the Nov. 1938, issue and appeared monthly. Notwithstanding their voluminous nature, the subject is not exhausted with the appearance of the Sept. 1939, issue (No. 360). The nature of slags is covered a little more thoroughly than in the series of articles appearing mostly currently in the same journal by Feret on slag cements. Aside from this the quality of these cements is covered in a somewhat similar manner to that used by Feret. Since this cement is but one kind of slag cement, it can be realized that Blondiau covers it more thoroughly than Feret, who attempted to cover all kinds of slag cements. The author shows considerable enthusiasm for this material and is inclined to make statements savoring somewhat of sales efforts. This kind of hydraulic cement has received quite a market in France and Belgium, and it might surprise some to know that its use is permitted in Paris for reinforced concrete structures, although it is composed of 80 per cent slag, 15 per cent anhydride, and 5 per cent of portland cement or quicklime, generally the former. It is required to meet the usual French specifications for Standard portland cement.

A study of concrete road mixers

D. ROESSLEIN, *Die Betonstrasse*, Vol. 14, No. 10, p. 209, Oct. 1939 Reviewed by A. U. THEUER

This is the concluding number of a series of articles and summarizes the results of an extensive study of German concrete mixers used for highway construction. A few of the more important criticisms voiced are: (1) Serious mechanical deficiencies in the guy anchorages and rigging of batch lifts. (Between 1933 and 1936, 40.2 per cent of all serious mixer accidents were in connection with this particular element). (2) Sharp edges and corners on these same elements resulting in slow or non-uniform (jolting) emptying. (3) The water measuring devices in many mixers failed to meet the requirements usually demanded of them. Also, many were found to empty too slowly. (4) The automatic timing clocks were seldom found in proper order. This condition is ascribed both to deficiencies in the clocks as well as the fact that such an element has been found inconvenient. In most cases the clocks are considered to be insufficiently protected against hand tampering.

Criticisms of a more general nature consist in (a) excessive vibration of some machines causing increased fatigue and accidents on the part of personnel; (b) undependable starters on motors resulting in a tendency never to stop them during rest periods; (c) attention of two men often required where one should do; (d) lack of protection against influence of weather.

A considerable discussion regarding the efficiency of the different types of mixers as regards concrete quality and uniformity follows.

Reinforced concrete tanks with hydraulic walls for the storage of hydrocarbons

CH. BERTHELOT, *LeGenie Civil*, Vol. CXV, No. 18, Oct. 28, 1939, pp. 322 and 323.
Reviewed by R. L. BERTIN

The author gives a complete description and details of a new type of tank claimed to be proof against leakage of the hydrocarbons stored therein—the Brice tank. It is so constructed that the reinforced concrete side walls and bottom are maintained saturated with water.

This condition is realized by embedding within the walls and bottom of the tank a thin continuous partition of porous terra cotta provided with intersecting vertical

and horizontal cells so designed that water can be made to fill them completely.

Special steel ties penetrating through this partition at the common corners of the individual units serve to bind the inner and outer reinforced concrete shells into a monolithic wall.

In operation, the cellular spaces within the wall are kept filled automatically with water, which saturates the inner and outer concrete masses and creates the desired hydrostatic pressures within the pores of the concrete.

Since water and the hydrocarbons stored in the tank are not miscible, the latter cannot penetrate the saturated concrete nor cross the sheet of water within the wall, neither can it displace the water since, even at equal level, the water pressure is greater than that of the content.

Additional advantages claimed for this type of tank are its insulating properties and its colorific inertia which reduce to a large extent the losses of its content due to "breathing."

The Summit avenue bridge

CLARENCE W. DUNHAM, *Civil Engineering*, Vol. 9, No. 11, Nov. 1939, pp. 639-642
Reviewed by J. R. SHANK

A two-span cellular reinforced-concrete continuous structure was built to carry Summit Avenue over the New Jersey Approach to the Lincoln Tunnel. This type of structure was selected because of having to carry a three foot sewer, whose invert was approximately twelve feet below the surface of the street, over the Approach, and because of architectural requirements. The cellular design was such that the sewer was housed out of sight in one of the cells where maintenance could be carried out.

Three cells about 9½ ft. square, independent of each other, were constructed continuous over a center wall support. The spans were poured integrally with the end walls but hinged over the center supporting wall. The bottoms of the supporting walls were square ended. The supporting walls were built independent for each of the three box spans and later, after the dead load was on, connected by means of poured concrete keys. The bottom slabs of the boxes, curved between the supporting walls to a radius of 180 ft. 1 in. on the intrados, were poured separate from the side walls of the boxes. To transfer the horizontal shear and diagonal tension to these webs the joints were saw toothed by hand and two 6 in. reinforcing trusses were constructed along each side-wall to bottom connection. The main tensile reinforcing was grouped at the side walls. The top slabs were poured integrally with the side walls. U-stirrups of ¾-in. round steel spaced throughout the side walls served for diagonal tension reinforcement and to help connect the bottoms and the side walls. These stirrups also served to take the straightening out component of the main tensile steel in the curved bottom slabs.

Two spaces about 11½ ft. between box sections were spanned by slabs resting on sloped faces on the upper corners of the boxes. On top of these slabs and the boxes an earth fill of a foot or two was placed, upon which the pavement was constructed.

Special care was taken in the design of details such as fillets and cross ribs, not too close spacing of reinforcing, steel corner reinforcing, steel hinges at the tops of the center supporting wall, pilasters for taking large load or reaction concentration, etc. A hood at the end of the box section served to remove earth pressure from the end wall and to furnish an access room for the sewer section.

Heat treatment of portland cement clinker

Journal of the Society of Chemical Industry, June 1939, Vol. 58, No. 6, pp. 203-213.
HIGHWAY RESEARCH ABSTRACTS

A basic hypothesis in the manufacture of cement is that the clinker, apart from some small residue of uncombined lime, reaches a state of chemical equilibrium in

the burning zone of the rotary kiln, but that equilibrium is not maintained on subsequent cooling. At the hot zone, the clinker consists of a mixture of solid crystalline silicates and a molten liquid containing all the alumina and iron oxide together with lime, a small amount of silica, and the greater part of the remaining minor components of cement. On subsequent cooling the solid crystals and the molten liquid behave as independent systems, there being no reaction between the two as would be necessary to maintain chemical equilibrium. The rate of cooling will determine if the liquid is converted into a glass or into a crystalline mixture.

It is desirable to know whether the physical properties of the cement are changed appreciably if the clinker contains glass or if it is completely crystalline. To determine this effect, a number of synthetic clinkers were prepared from samples of calcium carbonate, alumina, silicia and feric oxide of high purity mixed with water and compressed into briquettes weighing about 500 g., which were fired in a gas furnace. The resulting clinker was then ground and separated into two parts, and each part reheated to the original firing temperature for one hour. One portion was then quenched in water, producing a glassy clinker, the other portion was cooled slowly, producing a crystalline clinker. Samples of two commercial clinkers were also treated in this manner. The clinker resulting from this heat treatment was then ground in a small ball mill with a predetermined optimum amount of gypsum to the desired fineness.

The cements were then subjected to a number of tests to determine the effects of the heat treatment. Setting time was determined by the depth of penetration of a normal Vicat needle at different times. Soundness of the cements were determined with the Le Chatelier apparatus. Strengths were determined from 1:3 sand mortar cubes ½ in. on each side. Resistance to sulphate solutions, shrinkage, heat of hydration and other tests were also made.

From the statistical data presented, both tabular and graphic, the following conclusions were drawn:

1. The crystalline cements were easier to grind in the small scale mills, but it is uncertain if this holds true under commercial conditions.
2. Glassy cements give rise to no difficulty in controlling setting time, whereas some trouble was experienced with the crystalline cements.
3. The crystalline cements had the lower heat of hydration.
4. The glassy cements gave somewhat higher strengths at an age of 28 days.
5. There is some evidence that the shrinkage of glassy cements may be somewhat lower than that of the crystalline cements.
6. The glassy cements had a considerably higher resistance to attack by sulphate solutions than the crystalline cements.

The effect of the addition of finely ground inert material to concrete

SIR ROBERT CHAPMAN and PAUL ENGBERG OLSEN. *The Journal of the Institution of Engineers*, (Australia) Vol. 11, No. 8, Aug. 1939, pp. 263-278. Reviewed by J. R. SHANK

Numerous tests were made on cement mixtures, neat, as mortars, and in concretes in which varying amounts of fine inert filler material were added. The fine materials were crushed rock consisting largely of feldspar and quartzite, crushed rock largely limestone, crushed rock largely silica, and a clay.

Neat Mixtures—Tensile and compressive tests were made using six water to cement ratios by weight, viz: 0.25, 0.28, 0.30, 0.40, 0.50, and 0.60 with fillers added in five percentages: 10, 15, 20, 30 and 40.

The strength depends on the water to cement ratio regardless of consistency.

Density as determined by the weight of the briquettes depends on the water to cement ratio for the drier mixes and on the water to cement plus filler ratio for the wetter ones.

The consistency of the mix as determined by the Vicat normal consistency needle depends on the ratio of water to cement plus filler. With filler added, the mix has a putty-like character that will permit of easier working and placing than is indicated by the measure of the penetration of the plunger.

The addition of finely ground material, consisting chiefly of silica or limestone, up to as much as 50 per cent of the weight of the cement, when set, shows slightly less contraction than neat cement. A fine material of the nature of clay when added as a filler produces an excessive and dangerous amount of contraction.

Soundness pats of different water-cement ratios with as much as 100 per cent of filler added proved sound when the filler was silica or limestone. The clay filler pats were decidedly unsound.

A mix containing a filler is usually more economical, particularly when strength requirements are not high and a dry consistency may be used. The author exhibits a formula and a graph sheet for working out economical mixtures.

Mortars—Tests were made on 1:2 and 1:3 mortars having water cement weight ratios of 0.40, 0.45, 0.50, 0.60, and 0.70 with 25, 50 and 100 per cent of fillers added. Only the limestone and silica filler materials were used in the mortar tests. The results of importance from the strength tests is the repeated confirmation of the water-cement ratio law.

Tests were also made for cement +50 and 100 per cent filler added in comparison with cement +50 and 100 per cent sand added. A result of this latter group was that: Generally speaking the sand mixes were the stronger for the lower water cement ratios and the filler mixes were the stronger for the higher water to cement ratios.

A rule for consistency was deduced: "The total water to cement ratio required to bring the mix containing filler to any given consistency equals the water to cement ratio needed to bring the sand and cement, without filler to the given consistency plus the ratio of filler over cement, multiplied by 0.25 for the 1:3 mortars or by 0.33 for the 1.2 mortars."

Porosity or permeability tests indicated that in all cases tested (1:3 mixes, limestone and silica fillers) the addition of filler up to 100 per cent of the cement greatly reduced the porosity. With both fillers the addition of 25 per cent of filler was sufficient to reduce the leakage through the specimen to about one-tenth of what it was with the plain cement and sand mortars. Absorption tests indicated that the more filler added the less the absorption.

Any addition of fine material increases the amount of contraction. For 50 per cent filler the increase was 70 per cent.

The addition of filler to mortars made from sands with hard and rounded grains reduces the resistance of the mortar to abrasion when the proportion of filler to cement is more than 50 per cent.

Concretes—Mixtures using quartzite and feldspar materials were made up according to the Fuller Maximum Density Curve to which two fillers, silica and limestone were added. Two unadulterated mixes were made, one using 2 in. material in the proportion 1:64, cement to total aggregate and another using 1 in. material to 1:54. To the latter enough of each of the two fillers was added to bring this proportion to 1:64. A fourth mix was made up using the 1-in. material to which 25 per cent of the two fillers was added. The averages for the two filler groups showed for strength

2120, 2890, 3440 and 3500 p.s.i. and for slump, 1.3, 0.8, 0.12 and 0.12. The slumps for the limestone filler group were considerably less than those of the silica group. The strengths were about 90 per cent of the silica filler group. The adulterated mixes though much lower in slump packed more satisfactorily with less tendency to segregation.

Another group of mixes was worked out using 1-in. material according to the Fuller Curve to which 25, 50 and 100 per cent of the three fillers, silica, limestone and clay were added. Two water to cement ratios were used. The results appear in the table. The numbers 1, 2 and 3 apply to the fillers, silica, limestone and clay, respectively.

WATER: CEMENT RATIO BY WEIGHT

Particulars of Mix	0.75			0.85		
	Strength lb. per sq. in.	Slump inches	Cement Content lb. per cu. yd.	Strength lb. per sq. in.	Slump inches	Cement Content lb. per cu. yd.
Concrete—No Filler	2,060	2½	528	1,360	4¼	512
Concrete + 25 Per Cent No. 1 Filler	2,730	⅝	512	2,080	5	498
Concrete + 50 Per Cent No. 1 Filler	3,240	¼	498	2,620	1¾	486
Concrete + 100 Per Cent No. 1 Filler	3,380	¼	474	2,700	1¼	461
Concrete + 25 Per Cent No. 2 Filler	2,920	½	513	2,050	5½	497
Concrete + 50 Per Cent No. 2 Filler	3,140	½	497	2,280	2	484
Concrete + 100 Per Cent No. 2 Filler	3,050	½	472	2,560	1½	459
Concrete + 25 Per Cent No. 3 Filler	3,070	½	512	2,420	4½	497
Concrete + 50 Per Cent No. 3 Filler	3,090	Nil	496	2,660	¼	483
Concrete + 100 Per Cent No. 3 Filler	920	Nil	469	1,710	Nil	457
Concrete + 25 Per Cent Additional Cement	1,860	10	609	1,330	10	587
Concrete + 50 Per Cent Additional Cement	1,680	10	667	1,460	10	651
Concrete + 100 Per Cent Additional Cement	1,780	10	788	1,720	10	755

Two other series were tested similarly excepting that 1:2:4 and 1:3:6 arbitrary mixes were used. The advantages were less for the 1:2:4 mix and still less for the 1:3:6 mixes.

Rattler abrasion tests gave no greater abrasion for the slabs containing filler than for those with none.

Contraction tests made with the silica filler added in 0.0, 25, 50 and 100 per cent showed 0.000374, 0.000414, 0.000492 and 0.000591 contraction, respectively.

The addition of finely divided silica or limestone may be added economically to concrete mixtures when strength, density, permeability, workability and abrasive resistance are prime requisites. In general the shrinkage of curing is greater. Finely divided unburned clay should not be used.

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President's Address*

By F. E. Richart

It seems to be the privilege of the President of the Institute in making his annual address to take inventory of the state of the organization, to point with pride to any outstanding accomplishments, or to view with alarm the approach of unfavorable conditions. For a number of years members of the Board of Direction, who had witnessed the rapid decline in our membership from a peak value in 1929 of more than 2700 to a depression low in 1934 of 1154, have concerned themselves with finding means of recapturing our lost prestige and potential usefulness. About three years ago the policy was adopted of appealing to a larger group of prospective members through a shift of emphasis on the papers placed on our Convention programs and those published in the JOURNALS. It was decided that we had too many papers on research or on highly theoretical topics; these so-called academic authors were so prolific that their articles were continually slipping by the Publications Committee and cluttering up the JOURNAL. The crying need, it was felt, was for more articles on construction and design, on specifications and current practice, on the manufacture and use of concrete products, on architectural treatment of buildings, on building regulations and on contractors plant and organization. This conclusion was a logical one, inasmuch as our membership shows a dearth of contractors, specification writers, concrete inspectors, architects, products manufacturers, highway engineers, building designers, and the like.

A survey recently made by the Secretary reveals the following approximate division of member interests in the Institute:

*Made at the Institute's 36th Annual Convention, Chicago, Feb. 27-29, 1940, by the retiring president, F. E. Richart, Research Professor of Engineering Materials, University of Illinois, Urbana.

	Per Cent
Engineering Design	22.6
Construction	18.0
Inspection and Control	16.3
Cement	8.7
Aggregates	8.0
Architectural Design	4.4
Concrete Products	4.0
Ready Mix Construction	2.0
Auxiliary Materials	2.4

While these percentages are indicative only of the members' *principal* interests (first two choices), it indicates that more than half of our members are included in the first three groups, *Engineering Design, Construction* and *Inspection and Control;* also that our representation in other important groups is sadly lacking.

The emphasis on *practical* papers does not mean that the Institute is to become a trade association, concerned only with the development of concrete construction as a business proposition. If that were the case, we would surely lose many of our most valuable members, who contribute professional services with no view to material recompense. The conception is rather that the Institute should continue its contribution to fundamental research and development, and at the same time, furnish interpretation and restatement of principles which will "shorten the lag between research and practice," to quote the theme of Mr. F. T. Sheets, in his address before the Institute two years ago.

The need for especial emphasis on papers describing job practice is seen in the fact that despite our avowed policy in the selection of papers, a majority of those published in recent JOURNALS still deal with research and analysis. These are several reasons for this: the experimenter or analyst is trained to feel that his work is not finished until it has been recorded in written form, while the builder or manufacturer is content to let his product speak for him; and in many cases feels that the knowledge and experience he has acquired is a trade asset to be retained for his own use. To spend time and effort to give away one's stock in trade does not seem to be good business. These may be some of the reasons why the descriptive article on "how to do it" often fails to reach the JOURNAL.

Despite the foregoing remarks, a glance over the last volume of the JOURNAL reveals a very creditable coverage of the concrete field. Volume 35 contains 36 papers, 237 current reviews of papers published elsewhere and 44 short articles on job problems and practice. This list compares very well with the JOURNAL of ten years ago, when the Institute at the peak of its membership and with a budget nearly

twice the current one published 39 papers and committee reports and a large list of abstracts of current publications. The current department of "Job Problems and Practice" articles is a rather obvious outgrowth of the Institute's policy and serves to furnish the reader with a short, concise answer to a specific technical question, at the same time that it permits the man on the job to pass on some bit of useful information without the need of writing a formal paper.

The current papers, reviews and question-and-answer department all reflect another aim of the Publications Committee, the diversification of our information. By increasing the breadth of the field covered, the Institute can serve an increasing variety of interests among its readers. This is accomplished within out budget limitations through insistence on short and concise papers. Many authors, at first offended by requests to shorten and condense their offerings, will admit later that the quality and readability was improved by so doing.

The Board of Direction believes strongly in the principle that to capture the interest and support of an increased audience, the Institute must first give increased service. The work of our Technical Committees is directed toward this objective. Of about 25 such committees, the majority are charged with the preparation of reports or manuals of current practice on a specific phase of design, construction, specification, shop manufacture or architectural treatment. For a few of the lean years of the past decade our budget would not have permitted the publication of many committee efforts and consequently there was less urge for their production, but we are now reaching a status which will not only permit such publication but demands it as a part of the service the Institute is to give its membership.

Service on committees of the Institute is a labor of love, requiring loyal, unselfish, and cooperative effort and often a real financial sacrifice, all to the end that an authoritative document may be produced that will reflect the best current practice, as viewed by the leaders in the profession. Committee members are chosen with careful consideration; *first*, as to their ability, knowledge, experience and professional standing and, *second*, as to their willingness to cooperate and to finish the job undertaken. The second qualification is by no means the least important. Thus committee membership, while a distinct honor, is much more a real responsibility. The members of Institute committees are noted for serious-minded attention to the job at hand, with a general absence of commercial interest and of pressure groups to be found in many professional and trade organizations. Such honest study and collaboration is a sincere tribute to the recognized values and possibilities of the material, concrete, for no mean, fly-by-night

product could have held the interest and enthusiasm of the thousands of men who have served the Institute in the thirty-five years of its existence. In fact, after some twenty-five years of study in the field of plain and reinforced concrete, and mindful of the hundreds and even thousands of articles written annually concerning its nature and use, I am inclined to say that no other building material has offered such a variety of problems to the inquiring mind or has been subject to anything like this intensity of scientific research, practical experimentation and observation.

Among the committee efforts which have recently brought (or will soon bring) the fruits of their labors to the Institute members may be mentioned those dealing with vibration, precast joist floor systems, silo staves, reinforced concrete design handbook, a manual of concrete inspection, proportioning, mixing and placing, and standard building regulations for reinforced concrete. The last mentioned deserves special comment, since Committee 501, has presented its findings, in the form of recommended building codes over a period of twenty-five years or more. The work of Committee 501, taken in conjunction with the forthcoming Final Report of the Joint Committee on Specifications for Concrete and Reinforced Concrete, on which the Institute has an official representation of five members, and the Handbook of Reinforced Concrete Design developed by Committee 317, will undoubtedly have a country-wide effect upon building design in this material during the next decade.

Mention of specific committee accomplishments moves me to another observation regarding the role of the Institute in bringing concrete to its present stage of development. For years, the Institute by maintaining a high standard of professional and technical ability in its committee membership and in the authorship of papers, has contributed immeasurably to a minutely detailed knowledge of material and resulting structure, to the general acceptance of concrete in a multitude of uses and to the confidence which designers, builders and owners now place in the reliability of this material for huge and expensive monumental structures. In striving for the truth, the Institute has fought the battles of the consumer, including the designer and his client, the investing public; of the producer, the manufacturer of cement and concrete materials; and of the builder, the contractor or construction company. It is inconceivable that a service of this kind could have been rendered through commercial channels alone, and I think that I may properly say that the concrete industry and the public in general owe a huge debt to the Institute, as one of the very effective and important agencies in this great development. This is not a debt

to be expressed in millions or billions of dollars, but its existence might well be recognized by those most vitally concerned through increased support in the continuance of the Institute's work.

Unquestionably there is no end to the work the Institute must carry on. There will always be need for the digesting, coordinating, consolidating and simplifying of new methods, new knowledge. Most new concepts go through a cycle of experimental development, followed by a rapidly expanding stage of complex and intensive elaboration, then a final condensation and simplification. Such a chain of events accompanied the modern theories of concrete proportioning, of design of flat slabs, rigid frames, suspension bridges, and the study of wind stresses. We seem to be in the midst of the cycle with regard to portland cement. There is much for the Institute to do as a clearing house for the education of the engineer, the builder and the ultimate consumer.

As an indication of one's inability to foresee the trend and importance of the developments of the future let us go back only fifteen years and contemplate the extent of our knowledge then regarding such subjects as high frequency vibration of concrete, high pressure steam curing, the use of color in concrete, ready-mixed concrete, the hydration and chemical composition of cement, volume changes and durability, special cements such as sulphate-resisting, low-heat, high-early strength, etc., the placing of huge yardages of mass concrete, the development of architectural concrete, the analysis of frames, slabs and other structural members. It requires no prophetic vision to foresee that just as far-reaching changes will take place in the next fifteen years. What the changes will be is difficult to say. However, it seems safe to say that great advances may be expected in minimizing volume changes, with consequent increase in durability; in the production of concrete of uniform and high strength, with improved methods of control; in the development of architectural beauty in concrete, through the media of color, surface texture and ornamentation, and in refinements in structural design of buildings, bridges and other structures, through greater facility in analysis and design procedure.

There are many topics relating to the future of the Institute, and to that of the building industry upon which we depend, but they lead invariably to questions of political and economic trends which I cannot discuss here, even though they may be far more significant than the technical questions which engage our interest at this Convention. The construction industry is still sick; commercial, industrial and residential building is decidedly subnormal; highway con-

struction suffers from diversion of tax funds; public works, in the form of buildings, dams and power projects, while averaging about as well as in the fifteen years before the depression, would need to be doubled to make up the present shortage in private works. While the Institute gains in membership and income since 1934 have been encouraging and substantial (a membership increase from 1154 to 1616 as of last month), they parallel rather closely the increase in construction during that period, and are doubtless largely dependent thereon. Hence, the outlook of the Institute is closely allied with those factors which will promote or retard progress in the already long overdue program of heavy construction throughout the country.

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The Properties of Concrete Mixes*

By R. F. Blanks, E. N. Vidal, W. H. Price, and F. M. Russell†

MEMBERS AMERICAN CONCRETE INSTITUTE

SYNOPSIS

This paper attempts to assemble in exemplified form the known properties, factors, and relationships pertaining to concrete in the plastic state. Details of concrete mix-design procedure are not included, and reference is made to the properties of hardened concrete only when its requirements directly affect or predetermine certain properties of the fresh concrete.

The introduction includes a brief discussion of the concrete mix-design problem and a highly condensed account of the more important historical developments. The second section discusses the known properties of concrete mixes and related facts, and presents supporting data and information from the literature and from unpublished records.

INTRODUCTION

The Problem of Concrete Mix-Design. The design of concrete mixes is a process of combining economically with water several granular materials to produce a plastic, homogeneous mixture which will develop predetermined qualities in the hardened concrete. The "homogeneous mixture" is therefore composed of individual grains of relatively solid materials separated by enveloping films of water, and void spaces filled with water and entrained air. The individual

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†Engineers, Chief Engineer's Office, Bureau of Reclamation, Denver, Colorado. This paper was prepared primarily for the consideration of the A. C. I. Committee 613 and is a compilation of data and information needed by that committee in accomplishing its allotted task of formulating a "Recommended Practice for the Design of Concrete Mixes." Mr. Blanks is chairman and Mr. Vidal, member, of the Committee.

grains vary from extremely fine particles in the cement to progressively larger sizes through the sand and various sizes of coarse aggregate. The percentage of void space and the surface area of the different materials vary with the size of particles, and the ease with which these particles can be "fitted together" and mixed with water to produce a workable mixture which will retain its "homogeneity" throughout the processes of handling, placing and initial hardening, is in part dependent upon their shape, surface texture, gradation, and other characteristics.

Various schemes for proportioning the ingredients of concrete have been evolved to obtain maximum density, minimum void space, the most economical cement content, minimum surface area or other objectives which would, theoretically, insure the most desirable qualities in the resulting concrete. Even though some such schemes may be fundamentally sound, variations in any theoretically ideal conditions are usually required for the practical solution, because: First, the qualities of the hardened concrete required for the service conditions involved usually control the relative proportions of at least some of the ingredients, such as the cement and water; second, in order to secure economically the qualities required in the hardened concrete with available materials and provide the necessary degree of workability for the job conditions, proportions may be required which vary considerably from any theoretical combination; third, experience and service histories have demonstrated that satisfactory, high-quality concrete can be produced by combinations of aggregate particles varying between fairly wide limits, depending on the particular job and material conditions obtained: *Provided*, that the proportions of water to cement and the workability of the fresh concrete are appropriately controlled.

Present-day literature contains numerous proposed methods for designing concrete mixes, but none has been accepted as being generally applicable to all materials and conditions. The present consensus appears to be that the process of mix-design will probably never become an exact science because of the impracticability of predetermining many of the factors and influencing conditions in advance of job production. However, certain fundamental laws regarding the properties of mixtures of granulometric materials are well established. In addition, basic laws which control the physical properties of the hardened concrete have been determined, and empirical relationships have been learned from experience and investigation. With a thorough understanding of such available knowledge, the design of con-

crete mixes for any given set of materials and job conditions, such that the resulting concrete will closely approximate predetermined characteristics, becomes a relatively simple process. Such designed mixes, when actually applied to job conditions with plant-run materials, will usually require adjustment to comply with specified conditions and at the same time yield desired results as to placeability and economy.

From the preceding discussion, it becomes apparent that the problem of selecting and designing a concrete mix for a given purpose may be divided into four logical divisions:

1. Acquisition of a working knowledge of the known properties of plastic concrete and of those properties of hardened concrete which directly affect concrete mixes.

2. Assembly of all available information pertaining to the materials to be used, service conditions of the completed structure, job conditions, and specifications requirement; conducting investigations and tests as may be warranted, depending upon the size and importance of the job, to supplement the available information.

3. Design of mixes to start concreting operations by an acceptable and applicable procedure to approximate as closely as practicable the predetermined qualifications.

4. Adjustment of the selected mixes in the field as the work progresses to conform as closely as practicable to predetermined requirements under job conditions with maximum economy.

HISTORICAL

As a background to the discussions which follow, it is appropriate to review briefly the historical developments in the production of cementing materials.

Crudely burned lime and gypsum were used as cementing materials by the ancient Egyptians and Asiatics many thousand years before the Christian era, but its development from this stage was extremely slow and there is no historical proof of the use of hydraulic lime before the advent of the Greek and Roman civilizations. The Romans discovered that by intergrinding volcanic slag with burned lime they were able to produce a cementing material that would harden under water, and the relics of their structures that still stand attest to the excellence of their workmanship, which is also exemplified by the following quotation from The Water Supply Records of the City of Rome, written by Sextus Julius Frontinus, Water Commissioner in the year A. D. 97.

Repairs that can be carried on without cutting off the water of the aqueducts consist principally of masonry work, which should be executed at the right time, and conscientiously. The proper time for masonry work is from the 1st of April to the 1st of November; but with this restriction: that the work be interrupted during the hottest part of the summer; because moderate weather is necessary for the masonry properly to absorb the mortar, and to solidify into one compact mass; for the heat of the sun is no less destructive to masonry than is too violent frost. Nor is greater care required upon any works than upon such as are to withstand the action of water; for this reason, all parts of the work need to be done exactly according to the rules of the art, which all the workmen know, but few observe.

From the downfall of the Roman empire to the middle of the eighteenth century the art of making good cementing materials apparently was lost and it was not until 1756 that Smeaton in his search for building materials for the Eddystone lighthouse discovered that an argillaceous limestone when burned produced a lime that would harden under water. Later Parker found that by grinding the nodules of unslacked material rejected by Smeaton a more energetic hydraulic lime was obtained. Parker is given credit for the manufacture of the first natural cement. Natural cement was first produced in America in 1818.

Although the discovery of portland cement is usually credited to Joseph Aspdin during the period 1811 to 1827, his cement was far from a true portland cement and his work actually contributed less toward the scientific development of portland cement than that of several others. Natural cement rapidly gained widespread favor in the construction industry and greatly retarded the development of portland cement until the invention of an improved rotary kiln by Ransome in 1885. Very little natural cement is now used in the United States. Until the last ten years, efforts to improve portland cement were apathetic but at present there is a concerted movement among scientists and engineers to determine more accurately the constitution and behavior of portland cement and improve its composition, manufacture, uniformity, and service performance. [1, 2, 3, 4, 5, 6. *]

Since the discovery of cement, the concrete-making art has advanced generally, but at times with uncertain, floundering steps. The reason for the slow, spasmodic progress apparently is due primarily to apathy on the part of builders and engineers to take advantage of, and apply, available information. Such apathy has in turn been occasioned by the desire to obtain a false first-cost economy against the dictates of common sense and good judgment. Even at the present time there

*See references end of paper.

exists a much too-popular idea that concrete work can be cheaply accomplished with unskilled workmen and only superficial supervision.

As early as 1845 the necessity of proper grading of concrete aggregate was recognized by W. H. Wright in his Treatise on Mortars, in which he states that sands of various sizes may be mixed together with great economy of cementing matter; the fine sands, in this case, fill up the interstices of the coarser kinds, and greater density is obtained in the same volume. He realized, however, that a slight excess of mortar was required for proper workability. Wright made size analyses of sands to determine their relative value as an aggregate and recognized the superiority of weighing the ingredients over loose volume measurements. [7]

Machine mixing of concrete was employed as early as 1857 in constructing a bridge in Hungary. This mixer consisted of an open, revolving cylinder about 13 feet long and 4 feet in diameter, inclined at a slight angle. The stone and mortar were introduced through a hopper at the upper end of the mixer. [8]

In a book on concrete published by Henry Reed in 1868, it is recommended that the cement, sand, and coarse aggregate be properly mixed with the least amount of water for best results. He recommended that the concrete be placed by ramming in as dry a state as practical but points out that too dry concrete is as injurious as excessive water. [9] The use of dry concrete and rammed placing remained in vogue for over 50 years. The following quotation from an old specification of about 1890 typifies the "dry concrete" era.

> The concrete shall be formed of sound broken stone or gravel . . . to be mixed in proper boxes, with mortar of the quality before described, (one of cement to two of sand, by measure and a moderate dose of water) in the proportion of four parts of broken stone to one part of cement; to be laid immediately after mixing, and to be thoroughly compacted throughout the mass by ramming till the water flushes to the surface, the amount of water used for making the concrete to be approved or directed by the Engineer.

As the use of reinforcement became general practice in concrete work extremely wet mixes gained favor and came into almost universal usage as a result of the development of methods and equipment with which wet concrete could be cheaply "poured" in place. The following lines copied from an old specification convey some idea of the type of concrete used about 1910:

> The concrete shall be composed of cement, sand, and crushed rock in the proportion of 1:3:6. The concrete shall be used as wet as possible without being so wet that the matrix separates from the rock. Any places that are porous due to the matrix separating from the rock shall be removed. [10]

Empirical formulae such as 1:2:4, 1:3:6, etc., constituted the typically accepted method of proportioning concrete during the latter part of the nineteenth and the first part of the twentieth centuries. Such practices together with others implied by the specification quotations in the preceding paragraph reflect the lack of understanding of the fundamental laws and basic relationships governing the properties of concrete mixes, despite the fact that the French investigator, Feret, contributed an outstanding and substantial beginning toward scientifically sound concrete proportioning as early as 1890. Feret's classical papers, published between 1890 and 1900, are surprisingly modern even in the light of present-day concrete knowledge. [11]

Feret recognized the importance of plastic mixes, and derived from his tests with such mixes a relationship of cement and water to strength which is not essentially different from the present-day cement-water ratio law as may be seen from the following quotation from his paper: "For all series of plastic mortars made with the same cement and inert sands, the resistance to compression after the same time of set under identical conditions is solely a function of the ratio $\frac{C}{E+V}$, whatever may be the nature and size of the sand, the proportions of the elements—sand, cement, and water—of which each is composed." In the above law, E and V represent the volume of the water and air voids and C, the volume of the cement.

In 1907, Fuller and Thompson published their paper on the laws of proportioning concrete. [12] In this paper it is concluded that the larger size stone makes the strongest and densest concrete, that rounded aggregate gives denser concrete than broken stone, that concrete with angular coarse aggregate is stronger than with rounded coarse aggregate although less dense, that proper grading for density gives greatest strength, and that the grading curve of the best mixture resembles a parabola. Fuller and Thompson also concluded that the sand grading had more effect on the density and strength of concrete than the coarse aggregate grading and that the medium sizes of the coarse aggregate could be varied without affecting the strength of the concrete. It is worthy of note that these authors used mix proportions by weight and designed their mixes on the basis of absolute volume.

In 1916, Wig, Williams, and Gates published a paper [13] which concludes:

> There is no definite relation between the gradation of the aggregate and the compressive strength which is applicable to any considerable number of different aggregates. . . . The so-called maximum density curve, which is a combination of the

straight line and ellipse (Fuller's curve), does not represent the curve for maximum density excepting for the particular materials used in the tests from which it was derived, or very similar materials. . . . Concrete having a desired compressive strength is not necessarily guaranteed by specifications requiring only the use of certain types of materials in stated proportions. . . . Strength is just as much dependent upon other factors, such as careful workmanship and the use of the proper quantity of water in mixing as it is upon the use of the proper quantity of cement. . . . Too much emphasis cannot be placed upon the injurious effect of the use of excessive quantities of water in concrete.

In 1918, L. N. Edwards published his article on the surface-area method of proportioning [14] and in 1919, R. B. Young published a paper [15] discussing further the surface-area theory. Edwards' surface-area theory was founded on the assumption that:

The amount of water required to produce a normal, uniform consistency of mortar is a function of the cement and of the surface area of the sand aggregate to be wetted.

Continuing the thought, Young states:

The quantity required to wet the cement depends entirely upon the quantity of cement used, while the quantity required to wet the aggregate depends upon the total surface area of the particles making up the aggregate, and the latter is, in turn, dependent upon . . . its grading . . . If we assume the cement content and consistency to be the same, the conclusion then follows that the concrete which is made from the aggregate having the least surface area will require the least water in excess of that required to wet the cement and will, consequently, be the strongest.

It has been found that the amount of water necessary to produce "normal" consistency is dependent not only upon the quantity of cement and the surface area of the aggregate, but to a large extent on the proportion of voids and the character of the aggregate.[16] In the range of workable, practicable mixes, changes in surface area caused by changes in grading do not have proportionate effects on the water requirements of the concrete.

Abrams [17], in Bulletin 1, published in 1919 by the Lewis Institute, states:

There is the widest variation in the surface area of the aggregate without any appreciable difference in the concrete strength. . . . Our studies have clearly shown that surface area is not a satisfactory basis for proportioning aggregate . . . The sieve analysis curve of the aggregate may be widely different in form without exerting any influence on the concrete strength.

This bulletin proposes the "fineness modulus" for arriving at a satisfactory grading, and shows that for workable plastic concrete the water-cement ratio relationship was the most significant factor controlling the strength of concrete. Abrams showed that the water

requirements and workability were dependent upon the fineness modulus index of the particular aggregate and states:

Any sieve analysis curve of aggregate which will give the same fineness modulus will require the same quantity of water to produce a mix of the same plasticity and gives concrete of the same strength, so long as it is not too coarse for the quantity of cement used.

It is believed that Abram's water-cement ratio relationship is still the most satisfactory means of rating the quality of workable concrete and, because of the simplicity of application of Abrams' method of mix design, it did much to discourage the use of arbitrary proportions. Depending on the grading and soundness of the aggregate, low strength may be obtained with low water-cement ratios; however, high strengths cannot be obtained without low water-cement ratios.

Abrams published his paper on the "Effect of Fineness of Cement" in 1919. [18] Although this work was done before the turbidimeter came into use for measuring the fineness of cements, most of the statements made still hold. A few that pertain to mix proportions are listed:

1. There is no necessary relation between the strength of concrete and the fineness of the cement, if different cements are considered.

4. Fine grinding of cement is more effective in increasing the strength of lean mixtures than rich ones.

5. Fine grinding of cement is more effective in increasing the strength of concrete at 7 days than at ages of 28 days and 1 year.

20. One per cent increase in cement is more effective in increasing the strength of concrete in lean than in rich mixtures.

In 1923, Talbot and Richart published a paper [20] in which they found that the strength of sand-cement mortar was a function of the relation between the absolute volume of cement in the mixture and the total volume of voids in the mortar in place (water being considered as voids), and concluded:

For the usual concrete mixtures—at least, for those that make an easily worked concrete, including all mixtures used on construction—the bulk of the coarse aggregate is less than the bulk of the concrete; or in other words, the bulk of the mortar in a given volume of concrete is greater than the voids in the coarse aggregate alone —the term mortar being used here to include the cement, fine aggregate, and water. For such mixtures, the voids in the concrete may then be considered to be made up of the water and air voids in the mortar. The density of the mortar for the consistency used in the concrete is, therefore, an important factor in determining the strength in the concrete.

Talbot and Richart also found:

Aggregate graded to produce maximum density gave a harsh mixture that is very difficult to place and these minimum voids gradings of aggregate and even

the next finer ones, cannot be considered usable gradings for ordinary concreting operations.

In workable concrete of normal consistency the air voids may be neglected; therefore, Talbot's and Richart's cement-voids relationship may be considered for all practical purposes to give the same results as the water-cement ratio method for determining strength.

Many formulas have been derived, both empirically and mathematically, for determining optimum aggregate gradings. The one proposed by Bolomey in 1926[21] includes the cement as well as fine and coarse aggregate. His formula has been widely used in Europe.

McMillan published his book "Basic Principles of Concrete Making" in 1929.[19] He stresses the fact that handling, placing, and curing of the concrete may have much greater effects on the useful life and appearance of the structure than small variations in cement, water-cement ratio, and type, size, and grading of aggregate. The basic principles applying to fresh concrete as stated by McMillan are as complete, valid, and applicable today as they were when first presented. The summary of available information presented in the following paragraphs reveals no new *basic principles*, but more information and data have been developed which should lead to a clearer understanding of the application of these principles.[19]

Inge Lyse, in 1932[23], stated:

> For concrete having a given consistency and a given type and gradation of aggregates, the water content per cubic yard is nearly constant, and . . . the strength of the concrete increases in direct proportion to the increase in the number of cement particles in a unit of water. . . . By means of the straight-line relation between the cement-water ratio and the strength of concrete, the magnitude of the change in the cement content for a given change in the strength of the concrete is readily determined.

Weymouth's "particle interference" theory was first published in 1933. He determined mathematically the volume relationships between successive size groups of particles based upon the theory that the particles of each size group are distributed throughout the mass in such a way that the distance between them is equal to the mean diameter of particles of the next smaller size group plus the thickness of the cement films between them. "Particle interference" occurs between two successive sizes when the distance between particles is not sufficient to allow free passage of the smaller particles. Weymouth later presented in detail the basis for his theory of particle interference and showed mathematical relationships that exist between it

and the mortar-voids theory of Talbot and Richart, with an example of the application to a practical problem. [24, 25, 26]

It is predicted that the next few years will bring about the development of a still more rational and clearer conception of the basic requirements in proportioning materials for concrete mixes. There have been many recent indications of such "dawning light," notably T. C. Powers' paper on bleeding, published in 1939. [27] Present-day practice in considering the particle size distribution (gradation) of materials for concrete mixes is generally confined to the aggregate. It is becoming apparent that the coarser end of the grading curve is the least important and that the finer the particles, including the cement, the greater their influence on the concrete making properties of a given combination of materials.

It is realized that there have been many other investigators who have made valuable contributions to the advancement of knowledge pertaining to fresh concrete but space does not permit a complete review. The purpose here was merely to present sufficient historical background to serve as an adequate foundation for the statements and discussions which follow.

DISCUSSION

The importance of plastic, homogeneous mixes for fresh concrete cannot be stressed too heavily, as practically every variable considered in connection with fresh concrete leads directly to, or is intimately connected with, the consideration of workability. Since this property of concrete is of such primary importance, it is considered appropriate to open the discussion of the properties of concrete mixes with the subject of workability and consistency. The active ingredients of concrete are water and cement. These materials impart to concrete its useful properties, determine its suitability for the purpose intended, and the relative proportions of cement and water* used in the mix either control completely or exert a major influence on all the known properties of the fresh as well as the hardened concrete. The influence of aggregate on the properties of concrete mixes and finally, the manner in which the various ingredients combine in a plastic mix conclude the ensuing discussion.

Workability and Consistency. The intangible property, workability, may be defined as the ease with which a given set of materials can be mixed into concrete and subsequently handled, transported, and

*The term water-cement ratio, as used throughout this discussion, is the ratio of the water in the mix, exclusive of the water within or absorbed by the aggregate, to the portland cement, and for the Bureau of Reclamation data, is on a weight basis.

placed with minimum loss of homogeneity. Workability is dependent on the inherent properties of the concrete itself and is also controlled by the particular form in which placement will occur. For example, concrete that would be considered workable for a massive dam would be inconvenient for a slab and buttress dam and actually unusable in a thin reinforced wall or beam. Under present-day conditions of rapid transportation and placement, the ease with which concrete may be manipulated without loss of homogeneity is of prime importance.

The importance of the workability of concrete is demonstrated by the many devices developed for its measurement, the more important of which have been listed by Glanville[28] as follows:

(1) The Slump Test [29]
(2) Flow Tests
  a. American Type [30]
  b. Graf's Apparatus [31]
  c. English Type [32]
  d. German Type [33]
  e. Flow Trough [34]
(3) Penetration Tests
  a. Pearson & Hitchcock's Apparatus [35]
  b. Smith & Conahey's Apparatus [36]
  c. Graf's Apparatus [37]
  d. Humm's Apparatus [38]
(4) Drop Test [39]
(5) Mixer Test [40 69]
(6) Deforming Tests
  a. Deforming Cylinder Test [36]
  b. Remolding Test [41]
(7) Compacting Test of Faury and Lamare [42]

None of the methods measures all of the factors involved in the workability of concrete, the remolding test of Powers probably being the nearest approach. This test was developed on the basis that the best index of workability is the ease with which the concrete changes shape, and is designed to measure the effort involved in causing a fixed volume of concrete to change from one definite shape to another. The remolding test does not satisfactorily evaluate such factors as finish, segregation, and bleeding and requires careful, skilled manipulation. It has been found to be a useful tool in the laboratory but is not well suited for general field or routine control.

The consistency or fluidity to concrete, defined as the transient resistance to forces tending to change its shape, is an important factor of workability that can be measured with some degree of accuracy. The slump and flow tests respond primarily to variations in the water content of the concrete and serve as a fair measure of the proper amount of water to be used for optimum workability. Whether that optimum is relatively good or relatively poor, these tests alone do not tell and someone's judgment must be applied in conjunction with them.[43] The slump test is more generally used than the flow test.[29] It has been claimed that the reproducibility of results by the

slump test is poor, but it has been demonstrated conclusively that where reasonable care is used by the operator, such is not the case. Because of its simplicity, the test is widely used in concrete control in the field.

The slump test does not measure plastic qualities, particularly in the harsher ranges of lean and undersanded mixes, however, it can be made an accurate yardstick of workability when used in combination with visual observations as to rodability, segregation, finish, and appearance of the fresh concrete. The Bureau of Reclamation has found it convenient to rate the visual observations as follows: (This method of rating is somewhat similar to that recommended by Davis and Kelley.)[44]

Rodability: Bad, poor, fair, good, very good, excellent;

Segregation: Sticky, none, some, considerable, excessive, free water;

Finish: Either (1), bad, poor, fair, good, very good, excellent, or (2), wavy, sandy, smooth, wet;

Appearance: Crumbly, lean, undersanded, oversanded, rich.

The rodability of the concrete can be noted while making the tests. The plasticity of the concrete is clearly indicated by the type and manner of bulging of the slumped specimen, and cohesiveness by its failure to break or crumble when tapped with the tamping rod.

Predetermination of workability by slump and/or other measure is an important step in selecting a concrete mix for a given purpose. A mix that is too dry or one that is too wet for the conditions of handling and placing involved will almost certainly result in faulty construction and inferior service of the finished structure. Experience has indicated that, with vibration, the optimum slump should be from 1 to 3 in. for paving slabs or mass concrete and between 2 and 4 in. for various types of reinforced concrete. Where vibration is not used, the upper limits stated above will have to be raised from 1 to 2 in. If difficulty is encountered and it is impossible to place the concrete at these limiting slumps, it may be concluded that the concrete proportions should be adjusted. The minimum slump commensurate with desired workability requires the least amount of cement and water. In general, the wetter the consistency, the greater the tendency toward bleeding and segregation of the mortar matrix from the coarse aggregate, while a mix that is too dry for the particular conditions of placing involved will require additional care in order to avoid honeycombing and rock pockets. An excellent discussion of workability and consistency is given by Troxell and Davis in their recent book.[45]

Water and Cement. While cement is the least stable ingredient used in concrete and water contributes toward nearly all the undesirable characteristics of concrete as a building material, these constituents form the chemically active paste which directly affects every known property of fresh as well as hardened concrete. The ratio of water to cement, which determines the quality of the cementing matrix, controls the strength, water-tightness and durability, as well as other properties, of the hardened concrete. Specifications for concrete for a given purpose usually stipulate the maximum permissible water-cement ratio or the minimum allowable strength, or both, and therefore, a knowledge of the effect of water-cement ratio on the above-listed properties of the hardened concrete is essential to the predetermination of certain characteristics of the mix. Because of the above facts, A. C. I. Committee 613 has unanimously agreed that selection of the water-cement ratio is the first step in the design of concrete mixes.

It has been demonstrated conclusively that the most important factor influencing the strength of concrete is the water-cement ratio, that the strength of concrete is an inverse function of the water-cement ratio and that this relation is consistent and dependable. These facts have given rise to the "water-cement ratio law" which states that for plastic mixes, with suitable aggregate, the strength and other desirable properties of concrete under given job conditions are governed by the net quantity of mixing water used per unit of cement.

Several equations have been developed and advocated by different investigators as expressions of the water-cement ratio law. The best known of these expressions was developed by Abrams:

$$S = \frac{A}{B^{1.5x}}$$

Where:

S = Compressive strength in pounds per square inch

x = Water-cement ratio by weight

A = A constant of 14,000

B = A quantity which varies with the type of cement, type of aggregate, and age of test

The average 28-day strength, water-cement ratio relationship of 900 mixes made with aggregates from 38 different Bureau of Reclamation projects is shown graphically in Fig. 1.

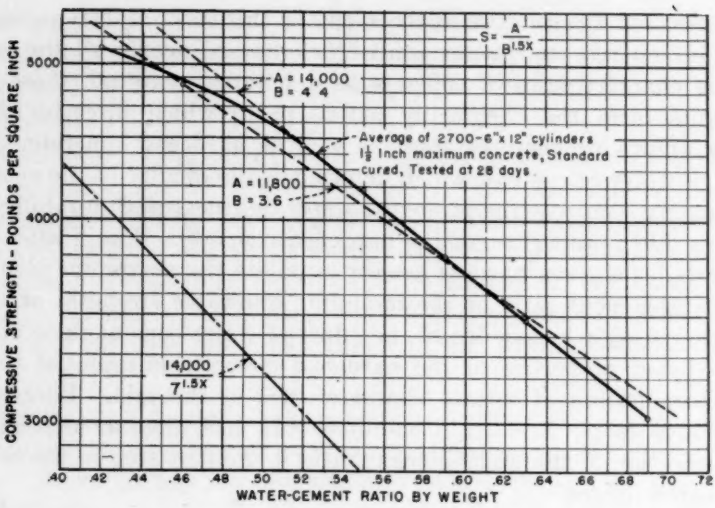

S = A / B^1.5X
A = 14,000
B = 4.4
Average of 2700-6"x12" cylinders
1½ Inch maximum concrete, Standard
cured, Tested at 28 days
A = 11,800
B = 3.6
14,000 / 7^1.5X
COMPRESSIVE STRENGTH - POUNDS PER SQUARE INCH
5000
4000
3000
.40 .42 .44 .46 .48 .50 .52 .54 .56 .58 .60 .62 .64 .66 .68 .70 .72
WATER-CEMENT RATIO BY WEIGHT

FIG. 1—RELATION BETWEEN WATER-CEMENT RATIO AND COMPRESSIVE STRENGTH. (COMPRESSIVE STRENGTH PLOTTED LOGARITHMICALLY.)

It is interesting to note that for water-cement ratios above 0.52 in Fig. 1, the straight-line relationship set forth in Abrams' equation applies reasonably well and if this straight-line portion of the curve is extended to a point where W/C equals zero, it will strike the ordinate at Abrams' value of 14,000. However, as the line has a definite curvature for water-cement ratios below 0.52, it appears that Abrams' formula for predicting compressive strength does not hold for all water-cement ratios and materials. It is believed that in the lower range of water-cement ratios the failure of concrete occurs, in part, in the aggregate while for the higher ranges failure occurs almost entirely in the paste. Each of the concretes made with the 38 different aggregates was studied individually to determine a value for Abrams' *A* and *B*. While Abrams[47] concluded that a value of 14,000 for *A* was invariable, the results of this study show that *A* is not a constant but a value that varies for different materials from 7,000 to 19,000 with an average of 11,800. On Fig. 1 is superimposed a straight line which most nearly approximates all the points on the curved line. This straight line was drawn from the ordinate where the water-cement ratio equals zero at the point where *A* equals 11,800, and shows plainly that for water-cement ratios between 0.4 and 0.7, a value for *A* of 11,800 is more applicable for the materials tested than 14,000.

Mention should be made of the more recent cement-water ratio law proposed by Lyse.[23] It may be said that within the limits of

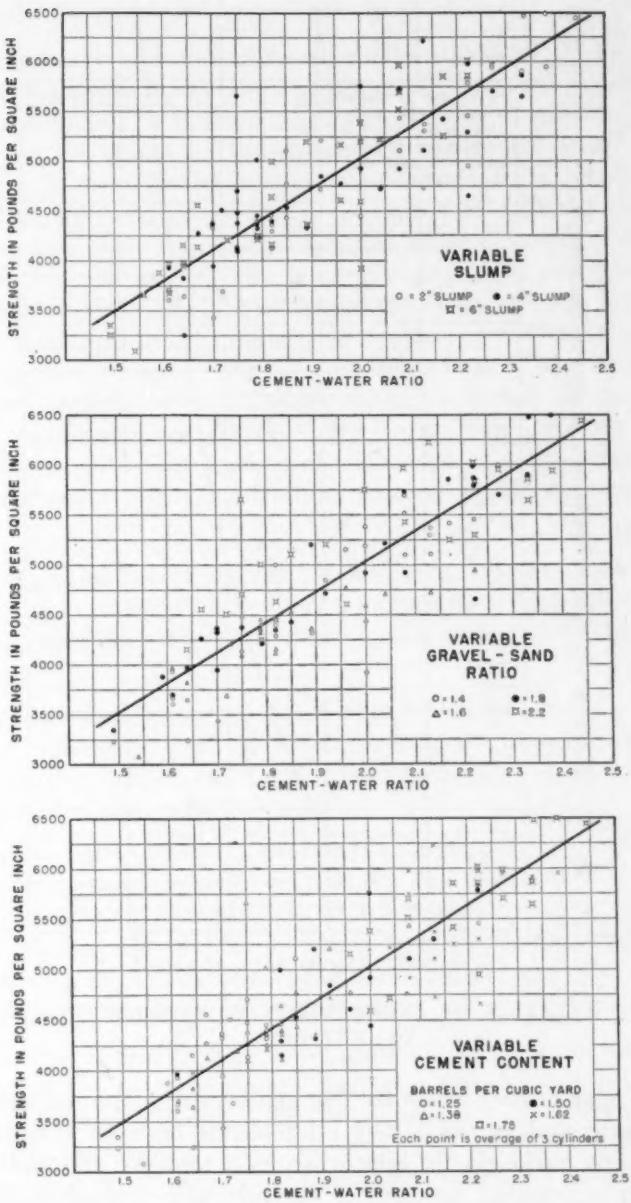

STRENGTH IN POUNDS PER SQUARE INCH
CEMENT-WATER RATIO
VARIABLE SLUMP
○ = 2" SLUMP ● = 4" SLUMP
⊠ = 6" SLUMP
VARIABLE GRAVEL - SAND RATIO
○ = 1.4 ● = 1.8
△ = 1.6 ⊠ = 2.2
VARIABLE CEMENT CONTENT
BARRELS PER CUBIC YARD
○ = 1.25 ● = 1.50
△ = 1.38 × = 1.62
⊠ = 1.75
Each point is average of 3 cylinders

FIG. 2—RELATION BETWEEN CEMENT-WATER RATIO AND STRENGTH FOR A TYPICAL NATURAL AGGREGATE. 1½-INCH MAXIMUM SIZE.

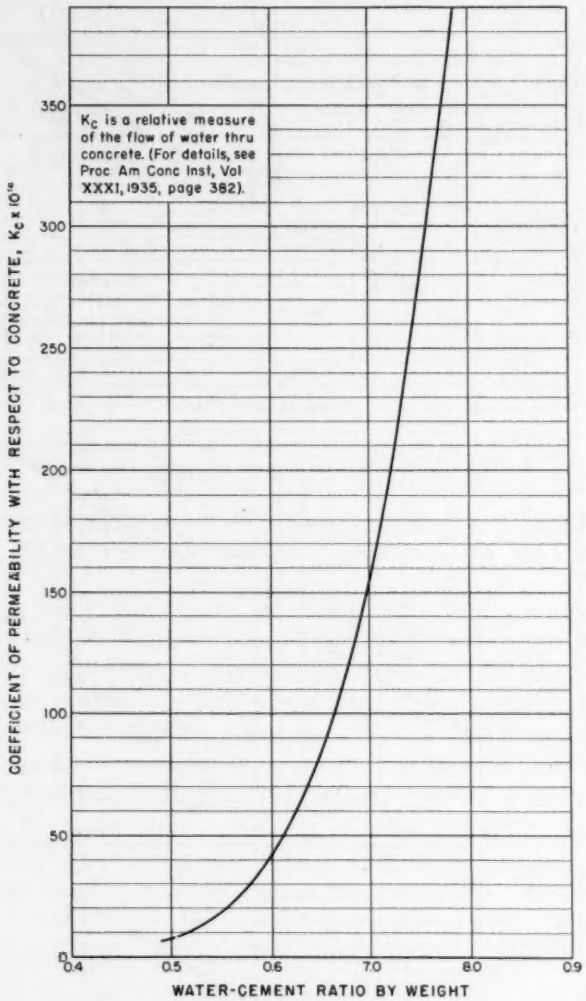


FIG. 3—RELATION OF PERMEABILITY OF CONCRETE TO WATER-CEMENT RATIO.

any particular project where the variables are a minimum (same type of aggregate and cement), the relationship between strength and cement-water ratio closely approximates a straight line and can be used with reasonable accuracy. The strength—cement-water relationship studied for one project, Fig. 2, shows that a straight line is probably the best relationship existing. The even distribution of the points in each of the three charts shows that within the range repre-

sented, the three variables, slump, gravel-sand ratio and cement content, have little effect on the relationship.

The durability of concrete refers to the length of its useful life, which ordinarily is limited by disintegration caused by various destructive agencies. Assuming that the aggregate, mixing, placing, and curing conditions remain constant, the durability of concrete depends to a large extent on the water-cement ratio. It has been shown that increased durability, as measured by freezing and thawing tests, is obtained by decreasing the water-cement ratio with a given mix, the slump and unit water content being correspondingly decreased; by decreasing the water-cement ratio with the slump and unit water content held constant, the cement content being increased accordingly; or by decreasing the unit water content and slump for a fixed water-cement ratio with accompanying reduction in cement content.[46] Watertightness and durability are in many respects synonymous and the pronounced effect of water-cement ratios, greater than 0.60 as shown by Fig. 3 is, therefore, particularly interesting.[48]

Fig. 4 shows the effect of water-cement ratio, cement content, and water content on the drying shrinkage of concrete, from which it may be noted that the shrinkage varies almost directly with the water content, regardless of how variations in the water content are obtained.

Table 1 lists recommended limits of water-cement ratios for various types of structures and conditions of exposure, and was developed after consideration of the factors discussed above together with a review of current good practice.

After the water-cement ratio has been selected, the proportions of cement to aggregate is dependent on the amount of water required to produce the desired workability and slump, which in turn is influenced by the grading and type of materials. In this connection, it is convenient to understand how changes in the properties of the fresh concrete influence the water requirements.

The curve shown in Fig. 5, representing about 1,300 mixes made with 38 different aggregates, illustrates the relationship between water content and slump. The curve shows a 7.5 percent decrease in total water required when changing from a 4-in. to a 2-in. slump as compared with only 4.3 percent increase for the change from 4-in. to 6-in. slump. For practical purposes, a 3-percent change in water will produce a 1-in. change in slump. Powers found that it was necessary to increase the water content of a particular mix by 21.5

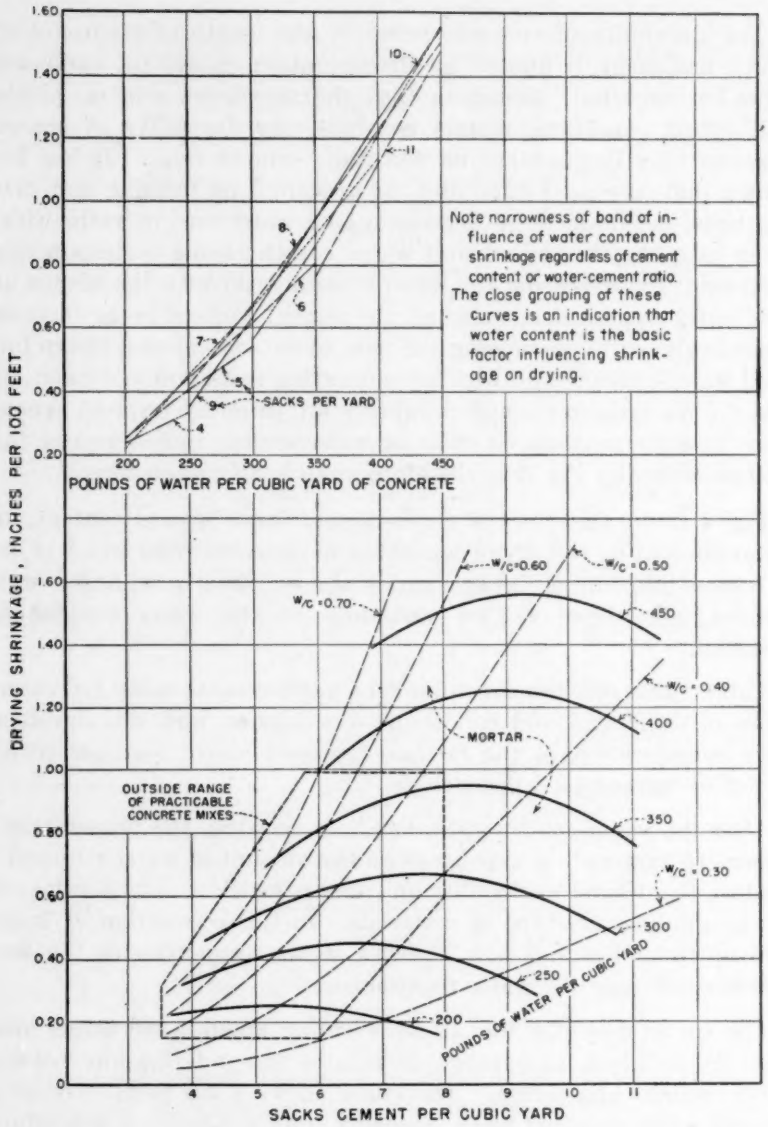

DRYING SHRINKAGE, INCHES PER 100 FEET
POUNDS OF WATER PER CUBIC YARD OF CONCRETE
SACKS PER YARD
Note narrowness of band of influence of water content on shrinkage regardless of cement content or water-cement ratio. The close grouping of these curves is an indication that water content is the basic factor influencing shrinkage on drying.
W/C = 0.60
W/C = 0.50
W/C = 0.70
W/C = 0.40
MORTAR
OUTSIDE RANGE OF PRACTICABLE CONCRETE MIXES
W/C = 0.30
POUNDS OF WATER PER CUBIC YARD
SACKS CEMENT PER CUBIC YARD

FIG. 4—INTERRELATION OF SHRINKAGE, CEMENT CONTENT, WATER-CEMENT RATIO, AND WATER CONTENT.

TABLE 1—RECOMMENDED LIMITS OF WATER-CEMENT RATIO

Type or Location of Structure	Water Conditions	Water-Cement Ratio, by Weight — Class of Structure: Thin Reinforced Sections, Walls, Light Structural Members, Exterior Beams, and Columns	Plain Thin Sections, Moderately Reinforced Sections, Tanks, Canal Linings, Dams of Thin Section	Plain Moderate Sections, Foundations, Piers, Heavy and Mass Section Dams
Hydraulic or water-front structures at water line or where intermittent saturation is possible	Sea Fresh Strong Sulphate	0.44 to 0.49	0.44 to 0.53	0.44 to 0.58
Ordinary exposed structures not subject to frequent wetting	.....	0.49 to 0.53	0.49 to 0.62	0.53 to 0.66
Continuously submerged structures—concrete completely submerged but protected from freezing	Sea Fresh	0.53 to 0.60	0.53 to 0.62	0.62 to 0.66
Pavement slabs on ground	Wearing Slab	0.49 to 0.53	0.53 to 0.58	.....
	Base Slab	0.58 to 0.62	0.62 to 0.66	.....
Concrete deposited under water	.....	0.49	0.49	0.49
Protected structures not exposed to weather, interiors of buildings, structures below ground	.....	0.66 Strength and w	0.53 orkability requi	0.74 rements govern

In each group the lower water-cement ratio applies to extreme climatic conditions, while the larger applies to climatic conditions that are moderate or mild.

percent in increasing the slump from 1 in. to 8.5 in., which is a fair check. He also found that the 21.5 percent increase in water increased the bleeding rate more than 2½ times, and that an increase of only 7 percent has a considerable effect on the bleeding rate.[27]

Fig. 6 shows that within practical limits, with constant aggregate grading, the unit cement content of a mix may be increased or decreased without materially changing the unit water content required to produce a given slump. McMillan found this relationship to be true.[19] Lyse also found that consistency remained almost constant regardless of the richness of the mix as long as the concrete was workable.[23] Fig. 6 does show that changes in the quantity of cement have more effect on the water requirement for coarse mixes and mixes having high slumps than they do for finer graded mixes or those having low slumps. However, the maximum change in water content for the abnormally large change in cement content of ½ barrel amounts to only 4 percent.

Fig. 7 shows that the cement content required to yield a mix having a given water-cement ratio and slump varies through wide limits

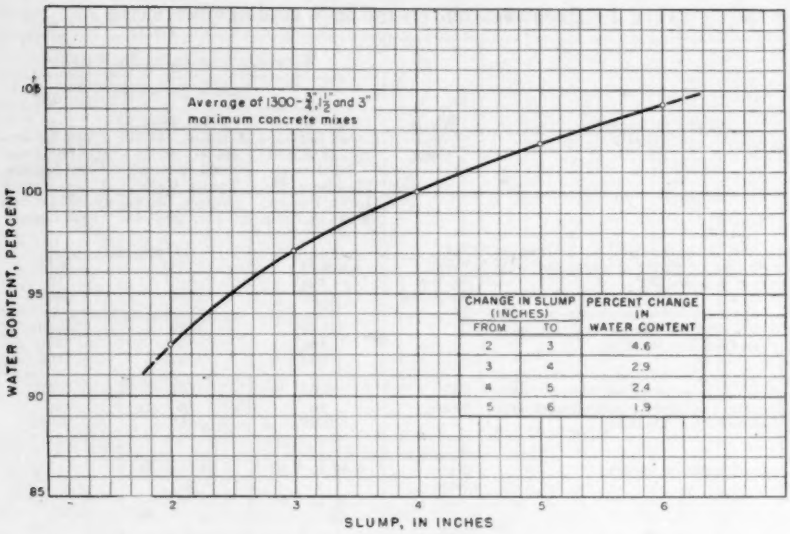


FIG. 5—AVERAGE CHANGE IN WATER CONTENT FOR CHANGE IN SLUMP.

depending upon the type, grading, and other physical characteristics of the aggregate and the brand and type of cement. From this it may be inferred that no arbitrary proportions would be applicable to all concretes.

In recent years there has been a growing tendency toward the manufacture of various types of cement, each particularly adapted

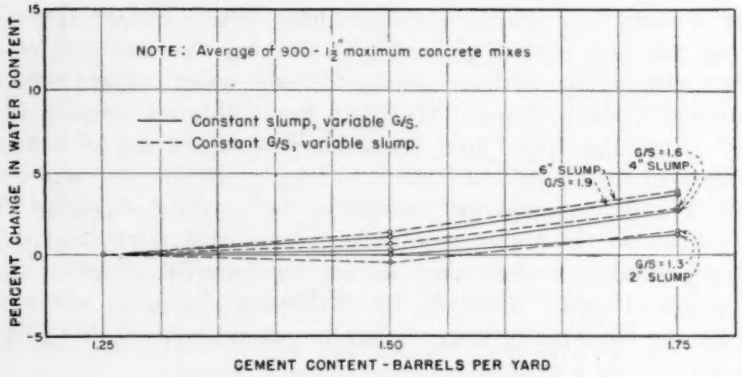


FIG. 6—AVERAGE CHANGE IN WATER CONTENT FOR CHANGE IN CEMENT CONTENT.

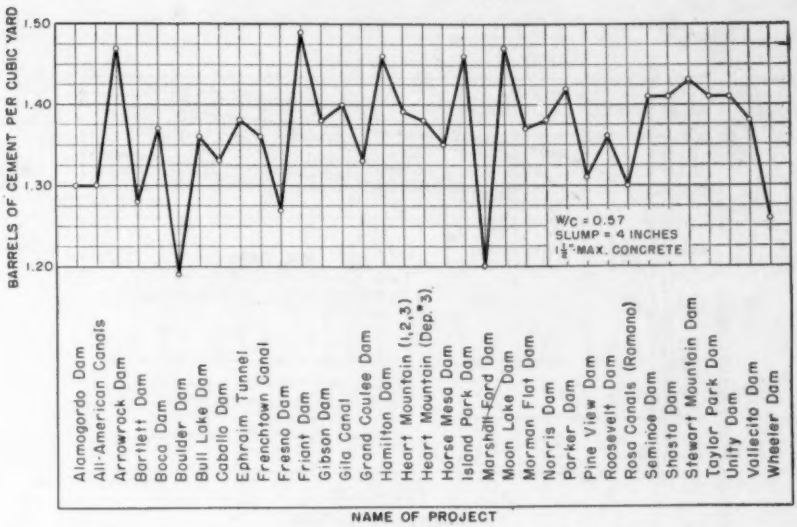


FIG. 7—EFFECT OF CHARACTER OF AGGREGATE ON CEMENT CONTENT.

for use under specific conditions. Type of cement is essentially a function of its chemical composition and the various types in common use can be conveniently classified in terms of the four theoretically predominating chemical compounds:

Tricalcium silicate: $3CaO.SiO_2 = C_3S$
Dicalcium silicate: $2CaO.SiO_2 = C_2S$
Tricalcium aluminate: $3CaO.Al_2O_3 = C_3A$
Tetracalcium alumino ferrite: $4CaO.Al_2O_3.Fe_2O_3 = C_4AF$

Table 2 shows compound composition data for five types of portland cement and Fig. 8 illustrates their comparative compressive strength under two curing conditions at various ages up to one year.

Increasing the fineness of a particular cement increases the workability and the strength at early ages, and increases the strength slightly at later ages, but a more rapid rate of heat generation results.[4, 46] Finer ground cement, within reasonable limits, also tends to reduce bleeding and segregation by making the mix more cohesive and requiring less water for a given consistency. Cements of the same type and fineness, but manufactured by different mills may have entirely different inherent bleeding characteristics. Cements coarser than about 1,800 square centimeters per gram as measured by the Wagner turbidimeter[49], contribute to bleeding and segregation and also require more water for a given consistency. Powers in his

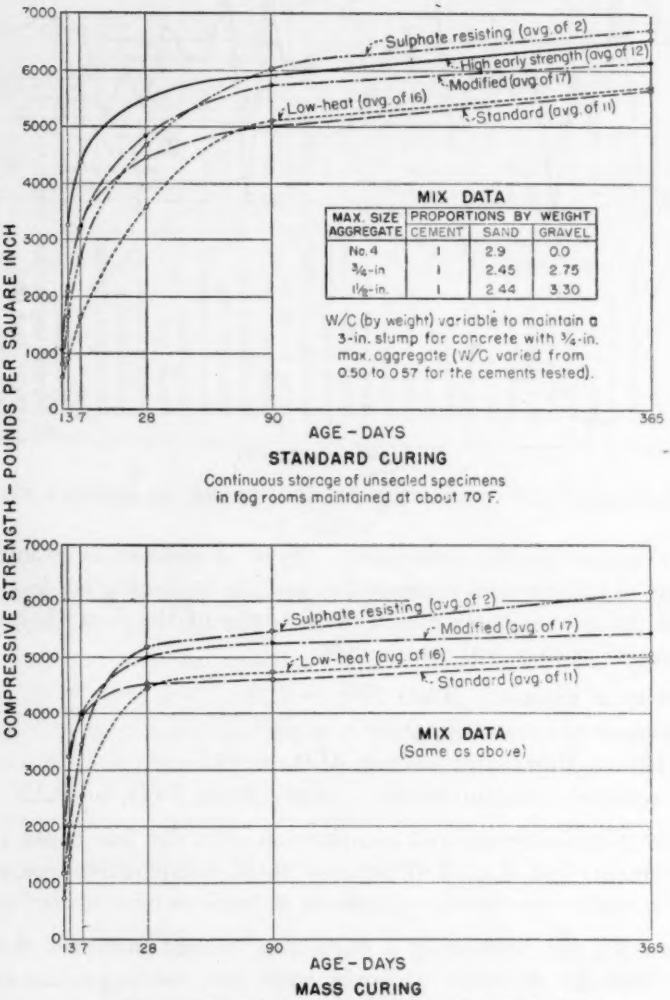

COMPRESSIVE STRENGTH – POUNDS PER SQUARE INCH
7000
6000
5000
4000
3000
2000
1000
0
Sulphate resisting (avg of 2)
High early strength (avg of 12)
Modified (avg of 17)
Low-heat (avg of 16)
Standard (avg of 11)
MIX DATA
MAX. SIZE AGGREGATE | PROPORTIONS BY WEIGHT: CEMENT | SAND | GRAVEL
No. 4 | 1 | 2.9 | 0.0
¾-in | 1 | 2.45 | 2.75
1½-in. | 1 | 2.44 | 3.30
W/C (by weight) variable to maintain a 3-in. slump for concrete with ¾-in. max. aggregate (W/C varied from 0.50 to 0.57 for the cements tested).
1 3 7 28 90 365
AGE – DAYS
STANDARD CURING
Continuous storage of unsealed specimens in fog rooms maintained at about 70 F.
Sulphate resisting (avg of 2)
Modified (avg of 17)
Low-heat (avg of 16)
Standard (avg of 11)
MIX DATA
(Same as above)
1 3 7 28 90 365
AGE – DAYS
MASS CURING
Specimens sealed in thin metal containers and stored in adiabatic calorimeter rooms, starting at 70 F, on a rising temperature cycle for 28 days and at 70 F thereafter.
Plotted points represent average results from tests with plastic mortars in 2-by 4-in. cylinders, concretes with ¾-in maximum aggregate in 3-by 6-in cylinders, and concretes with 1½-in. maximum aggregate in 6-by 12-in. cylinders.

FIG. 8—STRENGTH DEVELOPMENT OF VARIOUS TYPES OF CEMENTS IN MORTARS AND CONCRETES.

TABLE 2—COMPOUND COMPOSITION OF PORTLAND CEMENTS

Type of Cement		Compound Composition, Percentage								Number Averaged
		C_3S	C_2S	C_3A	C_4AF	$CaSO_4$	Free CaO	MgO	Ig Loss	
High Early Strength	Max.	70	38	17	10	4.6	4.2	4.8	2.7	
	Min.	34	0	7	6	2.2	0.1	1.0	1.1	
	Average	56	15	12	8	3.9	1.3	2.6	1.9	12
Standard	Max.	54	43	14	10	3.3	1.5	3.8	2.3	
	Min.	29	22	9	6	2.2	0.0	0.7	0.6	
	Average	43	31	12	8	2.8	0.8	2.4	1.2	11
Modified	Max.	50	46	9	18	3.3	1.8	4.4	2.0	
	Min.	29	22	3	10	1.9	0.1	1.5	0.5	
	Average	43	30	6	13	2.9	0.6	3.0	1.0	17
Sulphate Resisting	Max.	55	49	6	9	3.1	0.6	2.3	1.2	
	Min.	35	27	4	5	2.7	0.1	0.7	0.8	
	Average	43	40	5	7	2.9	0.4	1.6	1.0	3
Low Heat	Max.	33	61	8	18	4.0	0.9	4.1	1.9	
	Min.	10	41	3	6	2.5	0.0	1.0	0.6	
	Average	21	51	6	14	3.2	0.3	2.7	1.1	16

studies of the effect of the fineness of cement on bleeding found that paste made with cement ground to a fineness of 2,550 square centimeters could be combined with 50 percent more water than a cement ground to 1,540 square centimeters without increasing the bleeding rate above a permissible maximum.[27]

Tests conducted by the Bureau of Reclamation show the effect of fineness of various cements upon the water-cement ratio required to produce constant slump. Only a small number of tests are represented and the spread of individual values was large; however, when the water-cement ratios were averaged and plotted, the curve shown in Fig. 9 was obtained, which indicates a definite trend.

Aggregate. The influence of the aggregate on the properties of concrete mixes is controlled by the maximum size, grading, particle shape and texture, and other characteristics. It should be understood that cost and availability will materially affect selection for any particular job but for the purposes of this discussion, it will be assumed that such considerations are on an equal basis.

Maximum Size of Aggregate. The maximum size of aggregate to be used is controlled by the handling, mixing, and placing equipment, thickness of section, and the spacing of reinforcing bars. Satisfactory results have been obtained with concrete containing 9-in. maximum size aggregate for mass work, but Bureau of Reclamation experience has indicated 6-in. aggregate to be the largest size that can be handled

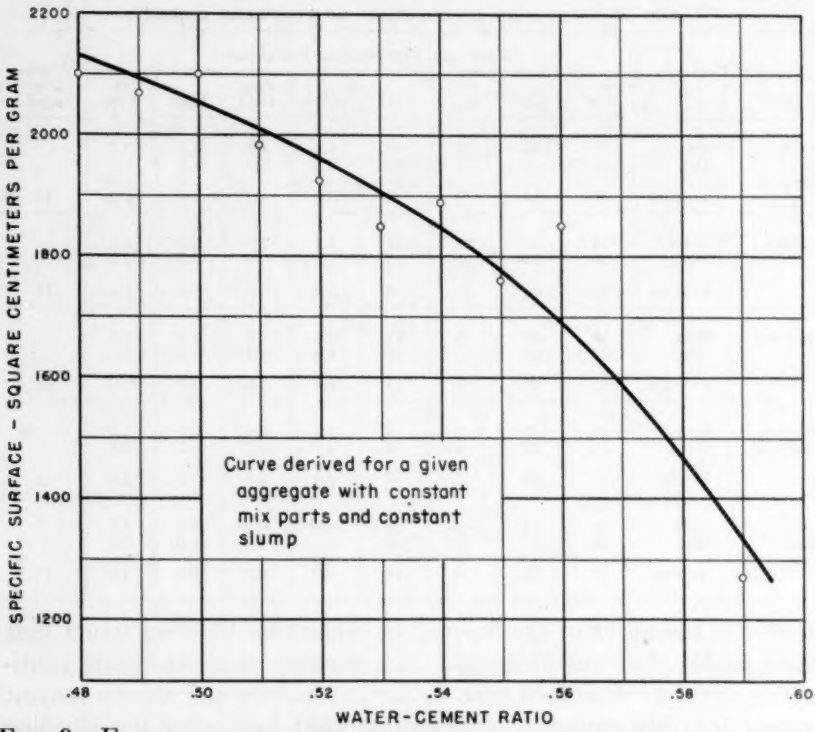


FIG. 9—EFFECT OF FINENESS OF CEMENT ON WATER-CEMENT RATIO.

efficiently in the mixing, transporting, and placing operations with the desired degree of uniformity.[50] The maximum size for thin, reinforced work seems fairly well established at not more than one-fifth of the narrowest dimension of the form or two-thirds of the maximum clear distance between bars.[51, 52]

Greater economy in cement requirement and higher quality may be obtained by using aggregate of the largest practicable maximum size. The reduction in both cement and water content, required for a given slump and water-cement ratio, occasioned by an increase in maximum size of aggregate, is demonstrated in Figs. 10 and 11. The higher degree of strength efficiency realized per unit of cement for successively larger miximum sizes of aggregate in concretes for a given slump is shown in Fig. 12. Gonnerman[55] states that the economy of cement increased up to a maximum size of about three inches, but beyond this point the use of larger sizes would be justified only if the cost of obtaining them would be offset by the saving in cement.

Gilkey[56] found that as the maximum size of aggregate was increased, other conditions remaining constant, the strength of the

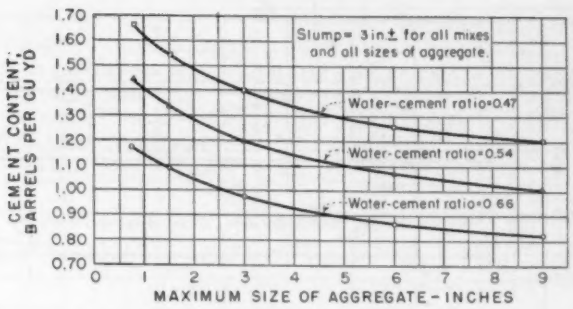


FIG. 10—EFFECT OF MAXIMUM SIZE OF AGGREGATE ON THE CEMENT REQUIRED IN CONCRETE.

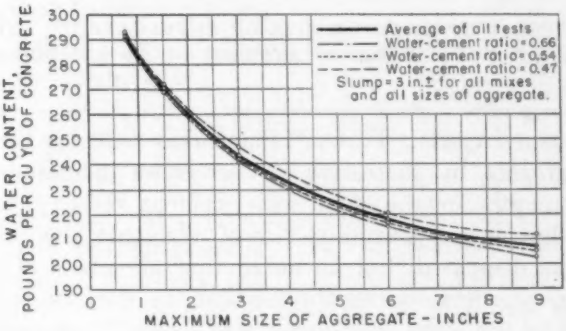


FIG. 11—EFFECT OF MAXIMUM SIZE OF AGGREGATE ON THE WATER REQUIRED IN CONCRETE.

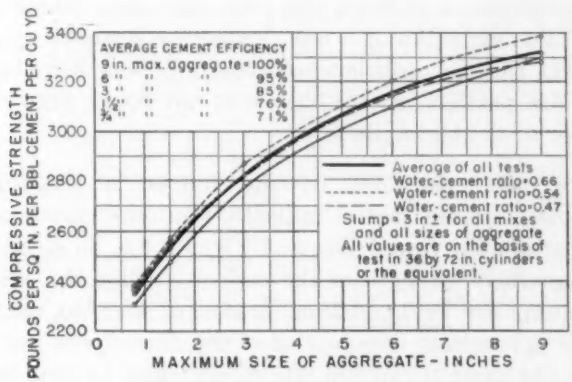

COMPRESSIVE STRENGTH POUNDS PER SQ IN. PER BBL CEMENT PER CU YD
3400
3200
3000
2800
2600
2400
2200
AVERAGE CEMENT EFFICIENCY
9 in. max. aggregate = 100%
6 " " " 95%
3 " " " 85%
1½ " " " 76%
¾ " " " 71%
Average of all tests
Water-cement ratio=0.66
Water-cement ratio=0.54
Water-cement ratio=0.47
Slump = 3 in. ± for all mixes and all sizes of aggregate
All values are on the basis of test in 36 by 72 in. cylinders or the equivalent.
0 1 2 3 4 5 6 7 8 9
MAXIMUM SIZE OF AGGREGATE - INCHES

FIG. 12—EFFECT OF MAXIMUM SIZE OF AGGREGATE ON THE EFFICIENCY OF CEMENT IN CONCRETE—AVERAGE OF 28- AND 90-DAY TESTS.

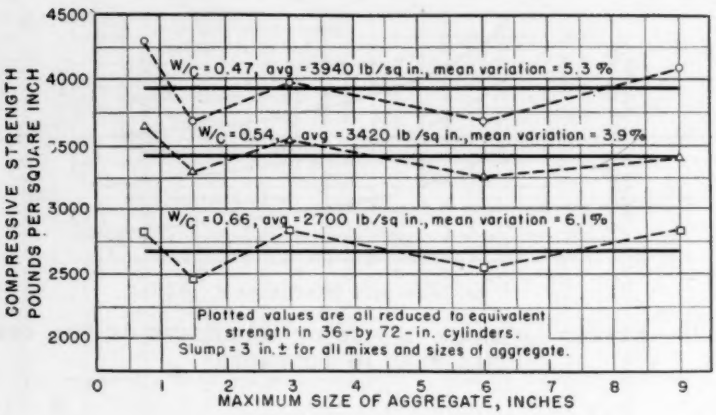


FIG. 13—EFFECT OF MAXIMUM SIZE OF AGGREGATE ON THE COMPRESSIVE STRENGTH OF CONCRETE—AVERAGE OF 28- AND 90-DAY TESTS.

concrete was decreased. Powers[57] found that reducing the maximum size of aggregate by successive wet-screening increased the slump and the strength for the same water-cement ratio. From this he concludes that as the maximum size of aggregate was increased the strength was decreased, but he noted that all mixes were tested in 6- by 12-in. cylinders and mentioned that the strength loss might be due in part to the effect of cylinder size. Lyse[58] comments that the average strength variations in Powers' data corresponds approximately to those due to the effect of cylinder size. McMillan[59] found that the strength of concrete was not materially altered by the removal of large pieces of aggregate from the mass provided that the removal of this aggregate did not change the consistency of the mix to the extent that an unbalancing occurred, provided also, that the maximum size particle in each case was not larger than one-quarter the diameter of the test cylinder.

Extensive tests conducted by the Bureau of Reclamation[53] to evaluate this variable factor indicate that the maximum size of aggregate, within the range shown in Fig. 13, has no significant effect on the compressive strength of the concrete, when the water-cement ratio and slump are held constant, provided, that the maximum size of aggregate is less than one-quarter of the diameter of the test cylinder. When the same maximum size of aggregate is used in a number of different sizes of cylinders, the indicated compressive strength decreases as the size of cylinder increases as shown in Fig. 14. The

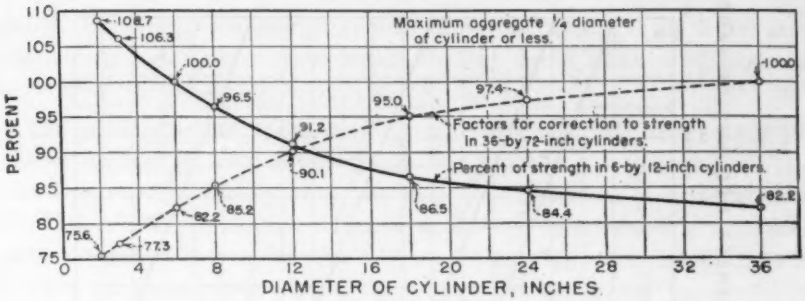


FIG. 14—EFFECT OF CYLINDER SIZE ON STRENGTH.

relative strengths obtained by wet-screening mass concrete and testing in various cylinder sizes were:[54]

6- or 9-in. maximum size concrete, tested in 18- by 36-in., 24- by 48-in., or 36- by 72-in. cylinders..... 77 percent

6- or 9-in. maximum size concrete wet-screened to 1½-in. maximum size and tested in 18- by 36-in., 24- by 48-in., or 36- by 72-in. cylinders.. 84 percent

And tested in 6- by 12-in. cylinders..... 100 percent

Aggregate Grading. It is generally agreed that the grading of concrete aggregate has considerable effect on the workability and related properties of concretes, but its effect on strength is still a point of controversy. This is due in part to the fact that concretes of

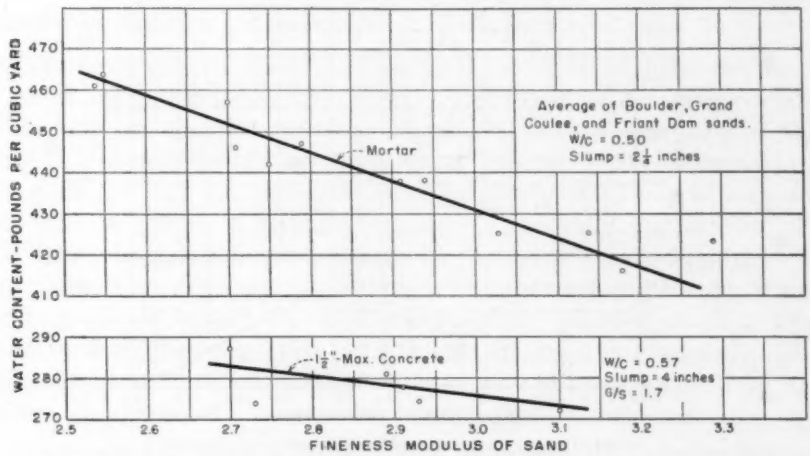


FIG. 15—EFFECT OF FINENESS MODULUS OF SAND ON WATER REQUIRED IN MORTAR AND CONCRETE.

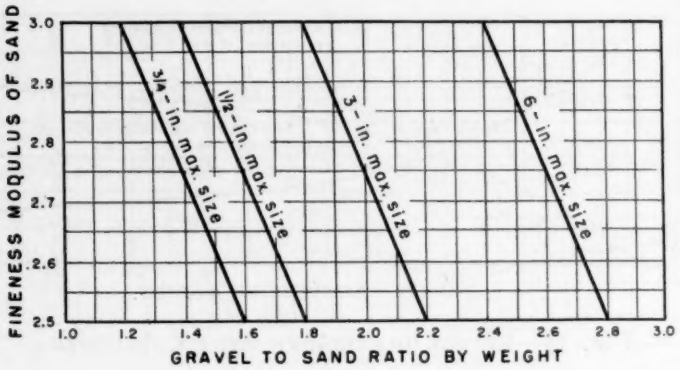
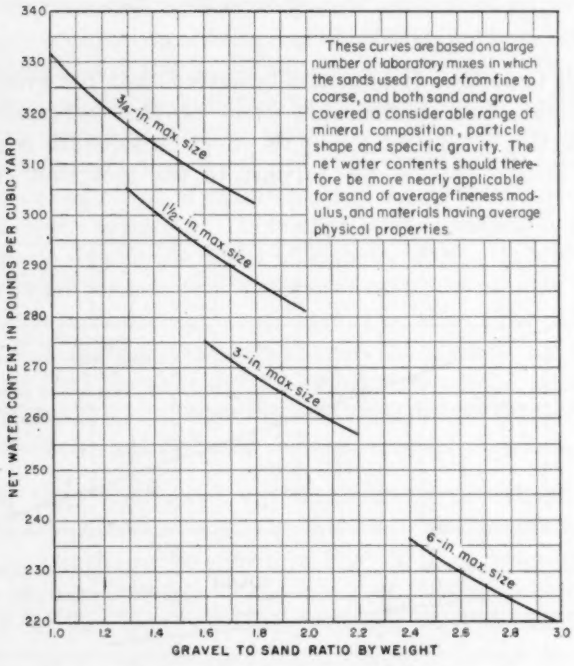


FIG. 16—CHART FOR SELECTION OF GRAVEL TO SAND RATIO TO USE WITH KNOWN OR ASSUMED FINENESS MODULUS OF SAND.



For the preliminary approximation of net water content for concrete having 4-inch slump, when the gravel to sand ratio is known or assumed.

FIG. 17—CURVES FOR SELECTION OF WATER CONTENT.

different aggregate gradings require different amounts of water and comparisons are not always made for the same quality of paste as measured by the water-cement ratio.

The percentage of sand, as well as its fineness and the maximum size of aggregate, has a direct influence on the water requirements. The influence of sand grading on unit water content is illustrated in Fig. 15. The percentage of sand necessary to produce a workable concrete depends to a large extent on its grading, and the shape and surface characteristics of the particles. An aggregate composed of smooth, rounded particles may give satisfactory results with coarser grading than would be permitted for an aggregate made up of sharp and angular particles with rough surfaces. Fig. 16 shows the approximate gravel-sand ratio (percentage of sand) to be used when the fineness modulus of the sand is known or assumed, within the limits represented. Any measure of workability requires judgment, and the relationships shown in Fig. 16 are based on experience in the construction of hydraulic works.

Fig. 17 illustrates the effect of gravel-sand ratio and maximum size aggregate on the unit water content of concrete. These curves are based on 1500 different laboratory mixes with natural aggregates from numerous sources, and any one aggregate will deviate somewhat from these curves. Experience has indicated that after the water content for any particular maximum-size aggregate and gravel-sand ratio has been established through actual trial, the water content for any other maximum-size or gravel-sand ratio can be estimated by drawing curves parallel to those shown.[46]

The grading of the sand has a very marked effect on the workability, uniformity, and finish of the concrete, and therefore has an important influence on the homogeneity, durability, and appearance of the finished structure. It has been found through years of experience that either very fine or coarse sand, or aggregate having a large deficiency or excess of any size fraction is generally less satisfactory for concrete than well graded material. Smoothly graded aggregate is preferred over gap gradings, although there are many instances where gap gradings have been used successfully. Natural sand graded within the following limits with the additional stipulation that the fineness modulus shall not be less than 2.50 nor more than 3.00 has been found generally satisfactory for specification purposes:

Tyler Standard Screen Size	Percentage Retained (cumulative)	Tyler Standard Screen Size	Percentage Retained (cumulative)
No. 4	0 to 5	No. 28	40 to 70
No. 8	10 to 20	No. 48	70 to 88
No. 14	20 to 40	No. 100	90 to 98

It is also desirable that no individual size fraction exceed 35 percent and that the material passing the No. 48 screen be at least 15 percent. Fluctuations in sand grading should be minimized on the job as uniformity is essential in the satisfactory field control of concrete.

The grading of the coarse aggregate may be varied through wide limits without affecting the workability of the concrete. Powers[41] found, with 1½-in. maximum size aggregate, that the ratio of the two coarsest sizes (⅜ to ¾ in. and ¾ in. to 1½ in.) could be varied over a wide range, without materially affecting the cement requirement or workability, provided the optimum percentage of sand and pea gravel (No. 4 to ⅜ in.) was maintained. Assuming that the optimum percentages of sand is used, workable concrete can be produced with natural coarse aggregate graded within the limits given in Table 3. It should be understood that the more uniform the aggregate grading is maintained for any given job, the less the adjustment that will be required in mix proportions and, in general, the more uniform will be the resulting concrete.

TABLE 3—GRADING LIMITS FOR COARSE AGGREGATE

Material	Screen Sizes (square openings inches)	Individual Percentages			
		¾-in. Max.	1½-in. Max.	3-in. Max.	6-in. Max.
Cobbles	3 to 6	0	0	0	16 to 32
Coarse gravel	1½ to 3	0	0	20 to 44	18 to 30
Intermediate gravel	¾ to 1½	0	40 to 56	18 to 36	16 to 28
Fine gravel	⅜ to ¾	50 to 70	24 to 40	14 to 30	12 to 22
	No. 4 to ⅜	30 to 50	16 to 30	10 to 28	8 to 16
	No. 4 to ¾	100	40 to 70	24 to 58	20 to 38

Fine sand will require more cement for a given water-cement ratio and slump than a coarser graded material. The effect is not critical and, within reasonable limits, has no significant influence on concrete strengths with constant water-cement ratio.

The grading of coarse aggregate may be varied through even wider limits than sand without producing appreciable effect on the cement requirement or strength of the concrete. Talbot[20] found that in workable mixes where the voids in the coarse aggregate were filled with mortar that the strength of the concrete was governed entirely by the strength of the mortar. Crum[60] explains that it is quite generally accepted that if the voids in the coarse aggregate are more than filled by the cement and sand mortar, the internal grading of the coarse aggregate will have little effect upon the strength of the concrete. Wig, Williams, and Gates [13] came to the conclusion

that for any aggregate there is no definite relationship between the gradation of the aggregate and compressive strength. McMillan[59], Glanville[61], and Kellerman[62] are of the same opinion.

In the preceding discussion regarding the effect of the grading of the coarse aggregate, it should be understood that the quality of each size of aggregate is considered uniform and that increase in strength due to the elimination of unsound particles in the aggregate or by changing the surface characteristics by crushing are not considered. Also, that no consideration is given to extremely fine or coarsely graded aggregate or to very lean mixes.

Particle Shape and Surface Texture of Aggregate. The actual effect of particle shape and texture of the aggregate on the properties of concrete remains quite controversial. Coultas[63] believes that the shape of the particle has an important bearing on the density of the mix and Gilkey [64] is convinced that thin or elongated materials otherwise suitable are a source of weakness in concrete. Feret[65] agrees, finding in his experiments with compaction that concretes containing elongated particles were noticeably weaker. However, Walker[66] states that flat or elongated particles, in such quantities likely to be encountered in practice, have no appreciable effect on workability.

Experience indicates that crushed aggregates do not produce as smooth and workable a mixture as materials having more rounded, smoother particles, grading, and other conditions being similar.[67, 61] Fig. 18 indicates that an increase in crushed aggregate requires an increase in water content to maintain a given slump, although some loss in workability may be expected, particularly for lower percentages of sand. Constant W/C was maintained in these tests, the required cement content increasing with higher percentages of crushed fragments as indicated in Fig. 19.

Investigations and experience in connection with the construction of Norris Dam reported by Tyler[68] demonstrated that the shape of particles produced by crushing in a specific mill had an important effect on the cement requirements. The superior mill which produced approximately cubical particles, resulted in a saving of five percent in cement requirement as compared with less satisfactory mill products.

It seems to be the concensus that an excess of unfavorably shaped particles (that is, thin, flat, elongated, flaky, splintery, etc.) will materially decrease the workability for a given water-cement ratio, and slump. To approach the workability possible with round pebbles, it is necessary to increase the cement content. Smooth, rounded

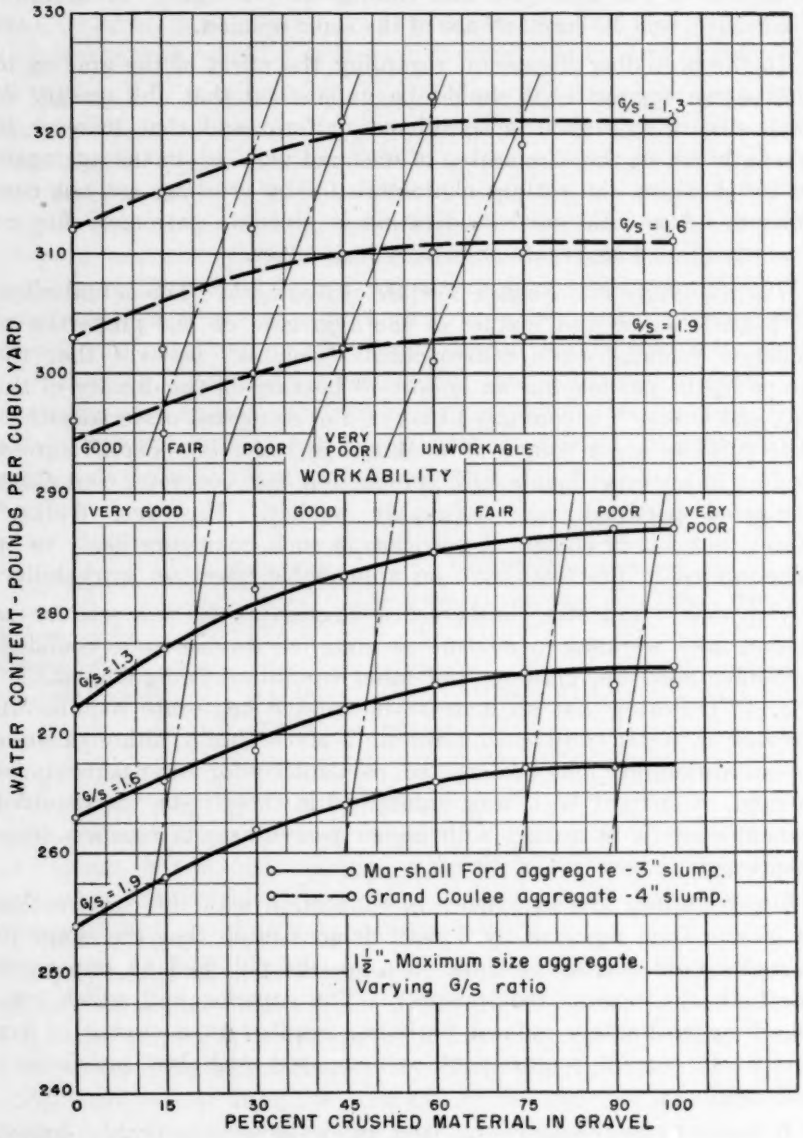


FIG. 18—EFFECT ON WATER CONTENT FOR VARYING AMOUNTS OF CRUSHED AGGREGATE.

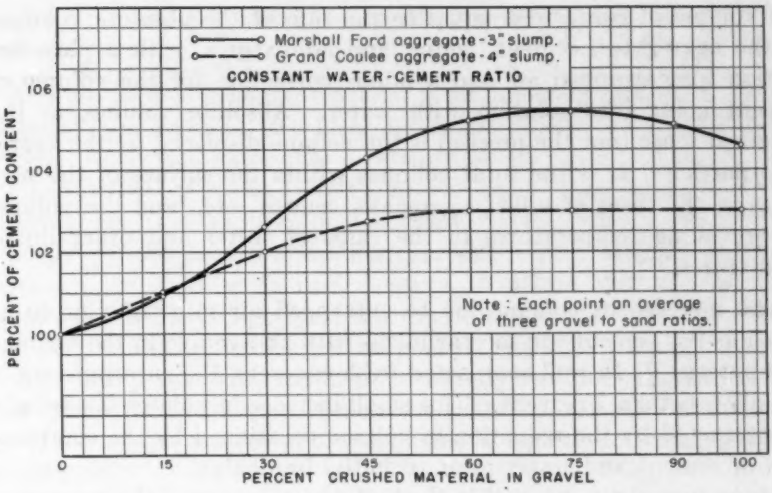


FIG. 19—EFFECT OF CRUSHED AGGREGATE ON CEMENT CONTENT.

particles are of advantage in that coarser gradings may be used, with consequent decrease in mortar requirement.

The surface texture may have a marked effect on the strength of bond with the cementing matrix. This is chiefly a matter of mechanical anchorage, sometimes due to roughness and often resulting from adhesion caused by suction of the paste into surface pores of the aggregate. Some types of seemingly smooth aggregate may therefore bond more strongly than ones with a rougher surface texture.

Other Physical Characteristics of Aggregate. While a knowledge of the specific gravity of an aggregate is necessary for accurate computation of concrete mix proportions, the specific gravity has no direct effect on the properties of the normal freshly mixed concrete. However, a natural or crushed stone aggregate with a specific gravity of less than about 2.50 should be investigated thoroughly before it is used in concrete, as a low specific gravity often indicates properties that may be injurious to the hardened concrete under certain conditions of exposure. Some of the properties often associated with low specific gravity are, porosity, softness, and high absorption, any of which may lead to breakdown under weathering action.

Fundamental Relationships. Kitts explains: "The actual or absolute volumes of aggregate, cement, and water are the basic physical measures of proportions of concrete because these fit together, physically, as absolute volumes, and in no other way. In other words,

the volume of concrete is equal to the sum of the absolute volumes of the aggregates, cement, water, and admixtures, with a plus correction for entrapped air and a minus correction for the volume of cement going into solution with water. Absolute volume, in the practical sense (not the precise) is the volume displaced by the aggregate particles. It is the total volumes within the surface of the particles in the cases of solids, aggregate, cement, etc., and the volume at prevailing temperatures in the cases of water and other liquid admixtures."[70]

The increase in volume due to entrapped air may amount to as much as 9.5 percent[71] in mortar mixes with an average in the vicinity of 3 percent.[72] In well compacted fresh concrete, the entrapped air is usually less than 1 percent. This small increase in volume is partially counteracted by the reduction in volume occasioned by the combination of cement and water prior to initial hardening.

Many investigations, within the limits of accuracy of the measuring equipment, have demonstrated that the volume of entrapped air is negligible in increasing the total volume of fresh concrete and could readily be disregarded. For nominal slumps and workable concrete, A. C. I. Committee 613 recognizes that for all practicable purposes:

$$A + C + W = 1 \quad \ldots\ldots (1)$$

where A = absolute volume of the aggregate
C = absolute volume of the cement
W = volume of the water.

Even though the relation of the particles to each other in a concrete mix is essentially one of absolute volumes and the analysis of a mix revolves about this relationship, it is impractical to proportion materials in the field in terms of absolute volume, and proportioning by bulk volume is not susceptible to uniform control. Weight proportioning of concrete materials satisfies all the basic absolute volume relationships and yet eliminates most of the practical difficulties in accurate field control because the absolute volume of a material is related to its weight and specific gravity as follows:

$$\text{Absolute volume} = \frac{\text{Weight of material}}{\text{Apparent specific gravity} \times \text{unit weight of water}}$$

Substituting weight and specific gravities for absolute volumes of concrete ingredients, formula (1) becomes:

$$\frac{W_a}{S_a \times 62.4} + \frac{W_c}{S_c \times 62.4} + \frac{W_w}{1 \times 62.4} = \text{volume in cubic feet}$$

where: W_a, W_c, and W_w = weights of aggregate, cement, and water,

respectively, and S_a, S_c, and 1 = specific gravities of aggregate, cement, and water, respectively.

A cubic yard of granular material is an indefinite quantity. If the material is moist sand in a loose condition, it may weigh only 2,000 pounds; or, if the material is a mixture of dry sand and gravel in a compacted state, it may weigh nearly twice that amount. This accounts for the familiar, but indefinite terms, "dry loose," "dry-rodded," "moist loose," and other expressions with which it has been necessary to qualify volume measure to attain any approach to exactness. A ton of aggregate, on the other hand, is a definite quantity and requires qualification only as to moisture content. As the weight of any concrete ingredient is directly related, through specific gravity, to the solid space which the material occupies in the concrete, the weight system of proportioning is logical, accurate, flexible, and simple.[46]

For any particular job using natural aggregate from one source, the specific gravities of the aggregate will seldom vary over 2 percent, and after an average value has once been determined, it usually can be used for all computations throughout the job.

By making use of the known fundamental and empirical relationships of concrete ingredients many rules can be derived that express relationships between various factors, such as water-cement ratio, slump, cement content, mix parts, etc., which may be used to advantage. Some examples follow: For a change in water-cement ratio (the aggregate grading and slump remaining constant), the total mix parts, including cement*, by weight, for all practical purposes are proportional to the water-cement ratios. Since a three-percent change in water content will change the slump approximately one inch, each one-inch change in slump (the mix proportions, by weight, remaining the same) is roughly equivalent to a 0.02 change in water-cement ratio, in the same direction. For each 0.02 increase or decrease in water-cement ratio without a change in the sum of the aggregates, the unit cement content is decreased or increased, respectively, by 0.01 barrel. There are many other such interrelationships of concrete ingredients that may be expressed in a number of different ways, depending upon the particular purpose, fancy, and preference of the individual. The nomographs shown in Fig. 20 have been found convenient for general use.

*Theoretically the ratio of the specific gravities of aggregate and cement (usually about 0.8) should be used in place of 1.0 parts of cement in the sum of the mix parts.

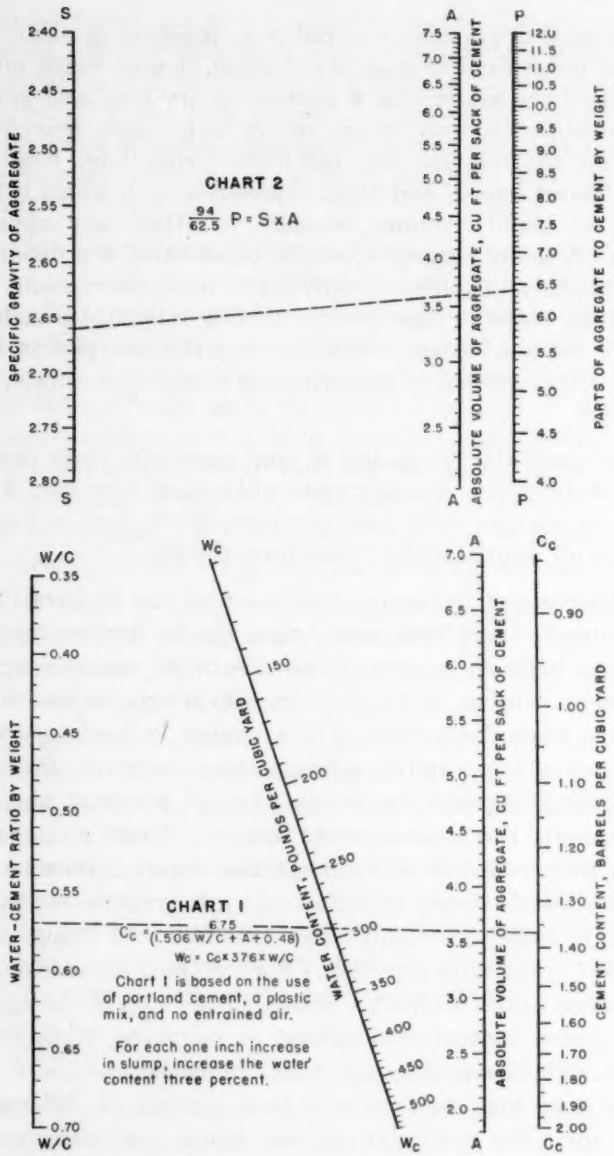


FIG. 20—INTERRELATIONSHIPS OF CONCRETE INGREDIENTS.

SUMMARY

1. Insofar as its physical characteristics are concerned, a concrete mix may be considered as a combination of individual particles of solid materials separated by enveloping films of water and void spaces filled with water and entrained air.

2. If the proportions of water to cement, which control the quality of the cementing paste in a concrete mix, and the workability of the plastic concrete are appropriately controlled, high quality concrete can be produced with various combinations of materials.

3. No theoretical procedure for designing concrete mixes has as yet been evolved which can be generally applied to all materials and job conditions, and the process of mix-design probably never will be an exact science because of the complexity of factors involved.

4. When full advantage is taken of available knowledge regarding the properties of concrete mixes and the effects of the various influencing factors, a concrete mix can be selected which will closely approximate predetermined qualifications for any given set of materials and job conditions.

5. Concrete mixes should be adjusted in the field as the job progresses in order to obtain optimum results with maximum economy.

6 A review of the historical developments pertaining to the proportioning of concrete mixes reveals a general lack of understanding of the basic principles involved as well as failure to apply available knowledge.

7. Poor practice in construction may exert greater influence on the useful life and appearance of concrete structures than small variations in the physical characteristics and proportions of the materials used.

8. Plastic mixes having suitable workability for the construction conditions involved is a primary requisite for good concrete work.

9. Consistency as measured by the slump test, when used in conjunction with observations on plasticity, harshness, and placeability, is the most practical method yet devised for controlling workability and maintaining uniformity in the field.

10. No concrete should be placed with a slump greater than four inches when vibration is used; without vibration the maximum will have to be increased about two inches.

11. For a given set of materials, the water-cement ratio is the most significant factor affecting the characteristics of the mix as well as the strength and other properties of the hardened concrete, and the

quality of concrete with given materials is inversely proportional to the water-cement ratio.

12. Predetermination of the appropriate water-cement ratio is the first step in the design of concrete mixes.

13. The type of structure and exposure to which the concrete is to be subjected determine in part the proper water-cement ratio to be used.

14. While Abram's water-cement-ratio-strength law does not hold precisely for all concrete mixes found in practice, it is the most preferred expression for predicting the strength of concrete.

15. Within the range of workable concrete and for a given mix and set of materials, each one-inch change in slump requires approximately a three-percent change in unit water content which is roughly equivalent to a 0.02 change in water-cement ratio by weight.

16. For the same slump and aggregate grading, the unit water content will remain constant for all practical purposes regardless of the richness of the mix.

17. To insure satisfactory workability of concrete and to prevent objectionable bleeding or water gain, cement should in general have an average fineness of not less than 1,800 square centimeters per gram (Wagner method).

18. Other factors remaining constant, as the fineness of cement is increased up to about 2,300 square centimeters per gram, the workability is increased and the water-cement ratio for a given slump is decreased.

19. The cement content required for a mix having a given water-cement ratio and slump varies through wide limits, depending upon the type, grading, and other physical characteristics of the aggregate and cement.

20. For a given aggregate, and assuming suitable gradations, the larger the maximum size of aggregate used in concrete, the lower will be the cement and water required for a given water-cement ratio and slump, and the greater will be the strength per unit of cement, all without loss of workability. Because of restrictions imposed by concrete-mixing and handling equipment, the maximum size of aggregate is limited to that passing a 6-inch screen.

21. For the same water-cement ratio and slump, assuming suitable gradations, the maximum size of aggregate has no significant effect on the compressive strength of concrete provided all sizes of aggregate are uniform in quality, surface texture, and particle shape.

22. Wet-screening the larger particles of aggregate from mass concrete has no significant effect on the strength of the resulting concrete.

23. The indicated strength of a given concrete decreases as the size of test cylinder increases.

24. For a given aggregate, the grading of both sand and coarse aggregate may be varied through rather wide limits without affecting the compressive strength of workable concrete, provided the water-cement ratio is maintained constant.

25. For a given aggregate, the percentage of each size of coarse aggregate can be varied through rather wide limits without materially affecting the workability or other concrete-making properties of the material. The "pea gravel" ($\frac{3}{16}$-$\frac{3}{8}$ in.) is the most critical size, and, therefore, must be controlled within narrower limits than the other sizes.

26. The grading of the sand has a more critical effect on the concrete-making properties of an aggregate, particularly on the workability, than does the coarse aggregate, but satisfactory workable concrete can be produced with various gradations of the same sand provided there is no serious particle interference.

27. In general, smoothly graded aggregates are preferable for economic production of concrete although gap gradings have been used successfully without loss of workability or strength.

28. The shape and surface texture of aggregate particles have a marked effect upon the concrete-making properties, particularly with respect to water and cement requirement and workability. A well rounded aggregate will produce more workable concrete with less cement for a given water-cement ratio and slump than a crushed aggregate or one containing considerable quantities of thin, flat, or elongated particles.

29. The basic measure for proportioning the materials in a concrete mix is the absolute volume of the individual materials.

30. The absolute volumes of concrete materials are functions of their respective weights and specific gravities and, therefore, weight proportioning of the ingredients of a concrete mix provides a practical economic, and fundamentally correct method for field use.

31. The total volume of fresh concrete is equal to the sum of the absolute volumes of aggregate, cement, and water (the assumption being made that no air is entrained in the mix).

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All activities of the Bureau of Reclamation are supervised by John C. Page, Commissioner of Reclamation, Washington, D. C. All engineering and construction is under the general direction of R. F. Walter, Chief Engineer and S. O. Harper, Assistant Chief Engineer, with headquarters in Denver.

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *SEPTEMBER 1940*

Discussion of a paper by Blanks, Vidal, Price and Russell:

The Properties of Concrete Mixes*

BY F. H. JACKSON AND A. T. GOLDBECK AND AUTHORS

CONVENTION DISCUSSION

F. H. Jackson†—This discussion is intended to supplement the information and data given in the paper entitled "The Properties of Concrete Mixes" by the Chairman of Committee 613 and his associates of the Bureau of Reclamation. According to the synopsis, this paper "attempts to assemble in exemplified form the known properties, factors and relationships pertaining to concrete in the plastic state."

The authors will agree, I am sure, that the discussion which they have prepared does not cover adequately that portion of the general problem covered by paving mixtures. Only incidental reference is made here and there to highway concrete, the chief emphasis being placed quite naturally on problems arising from the use of concrete in those types of construction with which the Bureau of Reclamation is chiefly concerned. However, it would appear that, in connection with "the acquisition of a working knowledge of the known properties of plastic mixtures and of those properties of hardened concrete which directly affect concrete mixes," adequate provision should be made to cover the requirements of all the major uses of concrete, including pavements, which, in 1938, accounted for about one-fourth of all of the portland cement shipped for domestic consumption in this country.

It is true that a concrete pavement is not as spectacular an engineering structure as certain other types—a large dam, for example. However, I believe I am safe in saying that, in general, a pavement is subjected to as severe as well as to as great a variety of destructive

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†Public Roads Administration, Washington, D. C.

forces as any type of structure in which portland cement is used. These include not only the stresses resulting from traffic loads but also those complex stresses which are the result of restrained expansion, contraction, warping, etc. Add to all this the stresses set up by weathering, such as alternate wetting and drying, alternate freezing and thawing, etc., and is it any wonder that premature failures sometimes occur? The wonder is that concrete pavements have given as good an account of themselves as they have when one considers the severity and the variety of the destructive forces to which they are subjected.

This discussion will endeavor to point out very briefly some of the requirements of pavement concrete which must be taken into consideration if maximum resistance to these destructive forces is to be provided.

In pavements, the critical stresses which result from the application of these forces are either direct tensile or bending stresses. Pavement concrete seldom fails in direct compression. It is for this reason that concrete mixtures for pavements are usually designed on the basis of flexural rather than compression strength as is the case with most types of structures. Therefore, all of the factors which affect flexural strength, including the quality as well as the quantity of the constituent materials in the mix, are of interest to the highway engineer in connection with a study of the properties of mixes.

Many series of tests have demonstrated beyond question that certain physical characteristics of aggregates other than grading and void content will very materially affect flexural strength. These differences seem to be associated with such characteristics of the aggregate as surface texture, porosity, etc., characteristics which affect the bond between the cement paste and the aggregates. This point was touched upon only incidentally in the paper under discussion but is being found to be increasingly significant in connection with the performance of road concrete.

In Mr. Blanks' discussion those characteristics of aggregate such as maximum size, grading, particle shape, etc., which affect quality primarily through their influence on the water-cement ratio have been stressed. Most of the principles which are stated are well recognized. Some may even be considered axiomatic. However, in the application of these principles to any particular type of construction, one must not lose sight of the fact that workability is a purely relative term. Thus, the paving engineer's conception of the term might not coincide at all with the ideas of other users of concrete. May I illustrate? On page 460, reference is made to Fig. 16, which is a chart

showing, for various maximum sizes of aggregate, the gravel-sand ratios to be used with sands of varying fineness. Although it is stated that this figure is based on experience in the construction of hydraulic works, I believe the fact that these values do not apply to paving mixes should be stressed. The ratios are entirely too high and if used would result either in lower quality or higher cost than necessary, depending upon the type of governing specification used. The point that I wish to make here is not that the values shown are wrong but that they apply only under one set of conditions. This and other charts in the paper are, I think, of value chiefly as illustrating the direction of certain trends rather than as specific tools to be used in a particular design problem.

The direct influence of aggregate type on resistance to tensile stress is particularly noticeable in coarse aggregates. There has been a tendency on the part of many users of concrete to conclude from Talbot and Richart's work that, so long as the voids in the coarse aggregate are filled with mortar, the strength of the concrete is governed entirely by the strength of the mortar. We have found that, insofar as flexural strength is concerned, this is far from true. However, our investigations have shown that, for a given combination of materials, the basic relation between strength and water-cement ratio established by Abrams holds for flexure just as well as for compression. In other words, although we cannot predict in advance the strength which will be obtained with any given water-cement ratio, we can determine very readily by experiment the water-cement ratio which will produce the required strength.

This we do through a very simple application of the well known water-cement ratio trial method of design. An example of the wide variation in proportions which results when a number of aggregates are evaluated in this manner will be sufficient. In this case, for the 25 aggregate combinations involved, the water-cement ratio required to give a modulus of rupture of 550 lb. per sq. in. at 14 days varied from a minimum of 0.64 for the poorest materials to 0.93 for the best. The corresponding cement contents varied from 7.1 sacks to 4.9 sacks per cubic yard. Although admittedly this is an extreme case, we have found frequently that among commercially competing aggregate the differences will amount to as much as one sack of cement per cubic yard, a difference of considerable economic significance.

Assuming that a paving mix is to be designed to produce concrete, which, with the required workability will have a specified flexural strength, the water-cement ratio trial method affords a theoretically sound procedure. It evaluates all of the factors involved, something

which no theoretical method based on grading considerations only can possibly do. So far as determining the optimum grading for workability is concerned, the final requirements will depend entirely upon the placement conditions involved. For trial mixes in the laboratory, a grading which will produce a plastic workable mix suitable for laboratory specimens can be used. Once having established the water-cement ratio, the optimum grading to use in the field can, in my opinion, be obtained best by trial on the job, using the materials actually furnished and the mixing, placing and finishing equipment available. I am thoroughly in agreement with the authors of the paper that the final adjustment in mix must be made in the field and must confess that I am still quite cold to the various more or less elaborate theoretical methods which have been suggested from time to time as the final answer to the problem.

In highway work, administrative officers usually insist on publishing prior to bidding the cement factor which will govern for each material combination. To obtain this information it is necessary to follow the selection of the water-cement ratio with a determination of the optimum mix for the required consistency which, for paving work, will be in the neighborhood of a 2½-in. slump for concrete to be finished in the usual way and a 1-in. slump for concrete to be finished by vibration. The basic rule which we follow has been stated many times by Abrams and others and is simply to use the coarsest grading possible and still maintain the necessary workability. This can be determined by eye in the laboratory with sufficient accuracy to establish the cement factor for bidding purposes. However, here again provision must be made for making final adjustments on the job, usually by varying the relative amounts of fine and coarse aggregate. May I emphasize that we are in complete agreement with the authors on this point?

I would like to close this brief discussion with a comment on the subject of concrete mix design in general. As I have said, the introduction of more or less complicated mathematical formulas and relationships into this problem leaves me completely cold. Is it not true that in the urge to develop interesting mathematical relationships on paper we forget that if the results are to be made usable they must be accepted by the profession as a whole? We must prepare and present our thoughts and recommendations on this subject as well as all other problems dealing with the use of concrete as a construction material in a manner which will appeal to the men who actually do the work. Otherwise they will not use them and we shall continue to delay closing what Mr. Sheets so aptly called the gap between theory and practice.

BY A. T. GOLDBECK†

A great deal of stress is laid in the paper on the high importance of the water-cement ratio in its effects on the strength of concrete and mention is made of the fact that the water-cement ratio relationship is not invariable, but that it is a different relationship, depending upon many factors such as cement, aggregate characteristics, temperature and others. There can be no doubt that the water-cement ratio has the most important influence on the strength, durability and permeability of concrete, but the very emphasis on and repetition of this fact, important as it is, has a tendency to make us lose sight of the fact that there are other very important influences on concrete properties which must be considered. One of these is the character of the coarse aggregate.

The report is based on gravel and is incomplete in not including the other two main coarse aggregates, stone and slag. The influence of the coarse aggregate on the strength and durability of concrete is often overlooked. We are prone to use compressive strength of concrete as the most important measure of its properties and perhaps there is an indirect relation between compressive strength and some of the other properties. But strength values are based on laboratory cured specimens which are most carefully cured under as nearly uniform conditions of moisture and temperature as we know how to control them. But what of the concrete in the structure? What effect does the aggregate have on the concrete as it is repeatedly exposed to shrinkage and swelling from moisture changes and to thermal expansion and contraction; to different changes in temperature from surface to interior of the mass; to freezing and thawing; to the effects of chemical action and perhaps to other influences? Our strength tests of quality of concrete give an indication of relative properties only provided ideal curing conditions exist. So let us not be governed too closely by any single water-cement relationship or set of such relationships in the face of these other influences.

Can there be any doubt that smoothness of the coarse aggregate influences the bond with the mortar, since in the course of time the mortar shrinks and swells under changing moisture conditions? Our own tests prove that roughness and the character of that roughness have great influence on the continuance of the bond strength and hence on a number of properties of concrete including durability.

Crushed stone concrete requires more mortar and a higher water-cement ratio than rounded aggregate concrete for given workability, but not necessarily more cement per cu. yd. to produce concrete of

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comparable suitability. One might jump to the conclusion that the higher water-ratio required by stone concrete automatically results in concrete of lower quality than with gravel using a lesser water-ratio. Any such assumption blindly accepts the water-cement ratio relationship as the all-governing one and utterly ignores the possibility of the aggregates playing any part whatever in influencing the important qualities of the concrete.

I am rather gratified to know that the permeability curve, Fig. 7, showing the relationship between permeability and water-cement ratio follows very closely Fig. 11 in our Bulletiu No. 10 on "Stone Sand." Our tests were freezing and thawing tests on stone sand concrete made with different water-cement ratios and at a W/C of 1.0 (by volume), rapid failure took place and this point on our curve corresponds to a similar point in Fig. 7 in the present report showing rapid increase in permeability. Apparently, here are two independent investigations which seem to check the statement in the report that there is a relationship between permeability and durability.

It is heartening to stone producers to know that, up to 6.8 to 7.0 gallons of water per sack of cement, the permeability is low and the durability of concrete is high. Stone concrete need not be made to suffer the penalty of a relatively higher cement factor in order to produce entirely suitable concrete from the standpoint of durability and permeability.

Tests for consistency or workability are sometimes used in a manner which imposes a burden on angular aggregates. In construction work a certain minimum degree of plasticity or workability of the concrete is necessary. It is important that the limitations of our laboratory equipment be recognized. We should not set up arbitrary laboratory methods for consistency or workability which may have no meaning in terms of the workability of the concrete in large masses. In a laboratory test practical placing conditions should be simulated, for otherwise we may fool ourselves rather badly. I refer to some of the devices mentioned among those used for workability tests.

Where the term "apparent specific gravity" is used, no doubt "bulk" specific gravity should now be substituted in accordance with the interpretation of the American Society for Testing Materials.

Mention is made of the possibility that curves similar to those in the report may be supplied by the National Crushed Stone Association applying to crushed stone. At present this cannot be done and may not be feasible with our limited facilities.

AUTHORS' CLOSURE

The authors are gratified that authorities in both the crushed stone and concrete pavement fields have contributed to the paper in such a beneficial manner. Their comments make worthy additions because they represent two fields whose primary interests are somewhat different from those of the authors, although all of us are interested in the manufacture of good, durable, economical concrete.

Mr. Jackson's discussion contains only one point on which there seems to be a difference of opinion. Mr. Jackson apparently designs his concrete mixes entirely on a flexural strength basis without setting an upper limit for water-cement ratio, and a water-cement ratio as high as 0.93 may be used if the resulting concrete has a flexural strength of 550 p.s.i. at 14 days. This is not an uncommon procedure in mix design for it is used by many designers where strength of concrete is the primary consideration. The authors' belief, however, is that strength alone does not measure the quality of concrete but rather that the water-cement ratio must be maintained below a certain maximum, regardless of the type of aggregate, to obtain a required durability. After a maximum water-cement ratio has been set by durability requirements, a cement factor could then be determined according to the method described by Mr. Jackson.

Mr. Goldbeck's discussion parallels Mr. Jackson's in that he uses strength as a measure of the quality of concrete and implies that higher water-cement ratios can be employed with crushed aggregate because of the higher strength obtained.

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Frost Resistant Concrete*

By R. B. Young†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

Most frost deterioration is due to a few preventable causes, of which the most common is segregation from undersanded and harsh mixes, improper placing, and overworking. Other causes are defective construction joints, and inadequate provision for drainage. Severity of exposure is an important factor in determining the amount of frost deterioration.

Frost resistant concrete is one made according to well recognized standards of quality in which, by adequate and competent supervision, more than average care has been taken with regard to those details that experience has shown to be the cause of frost deterioration.

The engineer with an operating company is in a different position than his professional brother who builds a structure and then moves on. Whether his structures are a credit to his skill or otherwise, he has to live with them and if, at times, this has its drawbacks, it also has its advantages for, if so inclined, he has an opportunity to study their behavior in service, to observe the effects on their performance of differences in materials and workmanship and to learn from them how to avoid mistakes on future work. The writer has had such an opportunity and it is the purpose of this paper to record some of the lessons learned from this experience as they relate to the resistance of concrete to frost action.

Before proceeding to a discussion of the causes of frost action or deterioration as applied to concrete, the meaning of the term should be defined. As commonly understood and here used, frost action is the breakdown of the structure of concrete caused by alternate

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freezing and thawing. The presence of moisture in the pores of the concrete is necessary thereto and this moisture may enter either by capillary action or pressure. The deterioration usually manifests itself by scaling and ravelling but it may occur without doing either.

Alternate freezing and thawing of concrete is a contributory rather than a fundamental cause for its deterioration. The fact that the temperature of concrete drops below 32° F. has no significance in itself, for dry concrete may be frozen and thawed repeatedly with negligible damage. There must be water in its pores before harm can result from freezing and it must be present in such an amount that when it increases in volume as it changes state, the ice formed will more than fill the pores containing it and so will develop disruptive forces.

One of the first lessons learned from a wide survey of concrete structures, is that only a very small percentage of the immense volume of concrete in service today, has defects of any importance. Even in those concrete structures where there has been extensive deterioration, and they are few, some parts are much more resistant to weathering than others and a great many are in first class condition. Since, in most cases, the same materials are used throughout a single structure, the variable condition of the concrete would seem to indicate differences in its quality that must have been due to variations in its processing or finishing.

Carrying our field observations further and making a careful analysis of the deteriorated areas found, it soon becomes evident that most of them may be grouped into relatively few types that keep repeating themselves on each of the structures examined. This, in turn, indicates a common or closely related cause for the defects of each type and a study of the conditions surrounding them, and their location in the structure suggests their probable cause. The significance of this was discussed at length in an earlier paper by the writer presented before the Institute.* The important conclusion therefrom, which bears on the subject of the present paper, is that the majority of these recurring defects coincide with an excessive porosity of the concrete at the points affected, brought about mainly by segregation.

Segregation may be described as the arch-enemy of concrete. By its very nature, it is a separation of the concrete mix into its component parts. In its grosser forms, the separation may be so com-

*"Lessons from Concrete Structures in Service", A. C. I. *Proceedings*, Vol. 25, 1929, page 64.

FIG. 1—DEFECTS IN CONCRETE STRUCTURES ARE COMMON AT CORNERS, EDGES, AND TOPS OF LIFTS. ALL THREE TYPES OF TROUBLE HAVE OCCURRED HERE.

FIG. 2—SEGREGATION RESULTING FROM DUMPING THE CONCRETE INTO THE FORMS AT ONE LOCATION.

plete that honeycomb results; in its lesser, there may be no visible defects, yet the concrete nevertheless is more than ordinarily susceptible to frost deterioration. Any segregation is thus objectionable from the standpoint of concrete durability.

The parts of a concrete structure that are particularly liable to deterioration because of segregation, are corners and edges where there is a tendency for mortar to gather, at the tops of lifts where "water gain" may occur and along fill planes and construction joints. The common practice of dumping concrete at one point in a form and pushing it around or letting it find its own way by gravity, is almost sure to cause segregation, especially the latter which results in the formation of both laitance and honeycomb pockets. Honeycomb is frequently found at the bottom of columns, the upper edges of horizontal joints, the underside of floor and deck slabs and even on vertical surfaces, due in most cases, to careless or unskilled methods of placing. One of the worst features of these defects is not the honeycomb itself, which can be easily patched, but the fact that, surrounding each of these areas, is a mass of concrete that appears sound yet which is partially segregated and thus liable to frost attack.

A less recognized type of segregation is caused by overworking. This may be from too much puddling next the forms, which causes a layer of mortar to accumulate there, or it may be due to over-finish-

FIG. 3—THE DARK AREAS OF THIS CONCRETE WHILE APPEARING SOUND HAVE DETERIORATED DUE TO PARTIAL SEGREGATION. NOTE THE HONEYCOMB IN THE CENTER.

ing which brings excessive fines and water to the surface. In either case, scaling results.

Scaling is also caused from excessive vibration near surfaces, but more particularly near horizontal surfaces where the segregation that accompanies over-vibration interferes with finishing. The cause of this scaling is apt to be overlooked unless the detailed history of the damaged concrete is known for it cannot be distinguished from scaling caused otherwise.

After segregation, the most common cause of dangerous porosity in concrete, is probably the use of harsh unworkable mixes. A mix

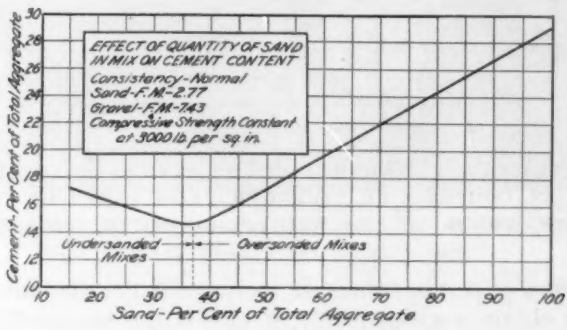

FIG. 4—A TYPICAL CURVE OF THE EFFECT OF SAND CONTENT IN A MIX ON THE AMOUNT OF CEMENT REQUIRED TO OBTAIN A CONCRETE OF GIVEN COMPRESSIVE STRENGTH AND WORKABILITY.

that tends to be unworkable is more difficult to handle than one which is not, and hence, is more liable to segregation. In addition, the factors that cause harshness in concrete mixes contribute to make them more than ordinarily porous.

Concrete mixes are harsh for one of several reasons, the most common of which are, first, undersanding or the use of insufficient mortar to carry the size and volume of coarse aggregate used, and second, a poorly graded fine aggregate, especially one lacking in the finer portions. This type of trouble is aggravated greatly where the mixes are lean in cement.

Undersanding is a relative term; it depends on the type and grading of the fine and coarse aggregate, the maximum size of the latter and the method to be used in placing the concrete. With the last two factors fixed, there will be, with any combination of aggregates, some ratio of fine to coarse, that will be found to require the minimum amount of cement to achieve a given result in terms of strength and

workability. (Fig. 4.) Any increase in the quantity of fine aggregate above this minimum ratio, might properly be considered as oversanding and any decrease as undersanding. The terms are thus used in this discussion.

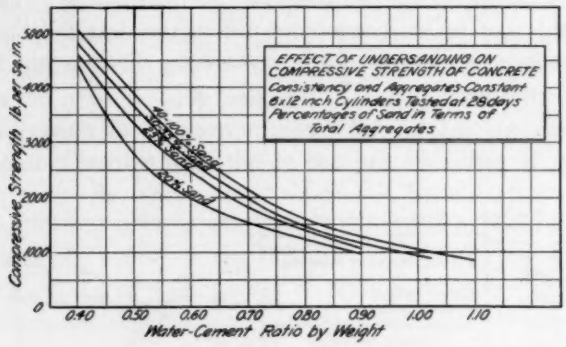

FIG. 5—A SET OF WATER-CEMENT RATIO CURVES ILLUSTRATING THE REDUCTION IN COMPRESSIVE STRENGTH FROM MIXES OF INCREASING HARSHNESS WHICH OCCURS WITH PROGRESSIVE UNDERSANDING.

Oversanding of a mix means an increase in the amount of mortar required to obtain a given result; it affects the workability, shrinkage and cement content of the concrete but has little effect on its resistance to frost unless the mortar is itself leaner than about 1:2½ when, with the coarser sands, it is possible to have undesirable porosity by reason of a lack of cement paste. Undersanding, on the other hand, cannot safely be tolerated for, if the sand content is decreased even a little below the optimum ratio, the mix rapidly loses workability. At the same time, the water-cement-ratio-strength curve begins to drop below normal unless means are taken to overcome the harshness of the mix by the method of consolidation. (Fig. 5.)

The practical application of these facts to the problem of frost resistant concrete is their bearing on its porosity. Any inherent harshness or lack of workability in the concrete mixes that cannot be overcome easily by the method of deposition used, increases the susceptibility of the concrete to frost action, partly as already indicated, by its greater tendency to segregation and partly, from lack of a sufficient quantity of fine aggregate to prevent particle interference by the coarse.

The ill effects of undersanding are common in older structures where the use of arbitrary proportions was the rule. A specification

FIG. 6—DEFECTIVE HORIZONTAL CONSTRUCTION JOINTS IN AN OTHERWISE EXCELLENT STRUCTURE.

FIG. 7—LEAKAGE THROUGH A DEFECTIVE HORIZONTAL CONSTRUCTION JOINT HAS DETERIORATED THE CONCRETE ADJACENT.

that required a mixture such as a 1:2½:5 or a 1:2:4, in which the sand portion formed only one-third of the total aggregate, would, if rigidly enforced, almost always result in an undersanded mix. The usual remedy in such cases was to add more and more water in an attempt to improve its workability. The results were generally unfortunate. Even today, undersanded concrete is still being produced in quantity, but present practice discourages the use of excess water and vibration provides better compaction, so that fewer examples of defective concrete from this cause may be expected in the future than have occurred in the past.

Harshness in a concrete mixture is influenced by the grading of the fine aggregate used. For equal results, a coarsely graded sand must be used in greater amounts than one finely graded. Where the concretes are produced under a specification that fixes the proportions of the aggregates, the likelihood of undersanding is thus greater with the coarse sand than with the fine. Most present day specifications permit the use of coarsely graded sands with as little as five percent passing the No. 50 sieve, and many engineers and architects even consider them as preferable to better graded materials but they should be used only where exposures are favorable and the proper corrections have been made in the proportions.

After segregation, the next most common source of frost deterioration is defectve construction joints and of these, more trouble occurs at horizontal than at vertical joints. The former have apparently presented less difficulty to designers and builders than the latter, which seem to be particularly hard to build so that moisture will not enter. They can be made watertight but, under ordinary job conditions and even with the best intentions on the part of all concerned, a large number of them prove defective in service.

A defective construction joint, whether horizontal or vertical, subjects the surrounding concrete to a particularly severe exposure because of the fact that the moisture movements through it, saturate the concrete adjacent. Thus, even where the concrete next the joint is of good quality, deterioration will occur in time and gradually spread. The writer has watched, year by year, joints in which this was happening, joints where little or no active seepage existed but where the adjacent concrete was slowly deteriorating by reason of repeated freezings and thawings while saturated with moisture sucked from the joint. A great many examples of this sort of trouble are to be seen in such structures as retaining walls, dams, reservoirs and tanks, etc.

Closely related to joints, are cracks that occur in the concrete from one cause and another. The ordinary shrinkage crack, as found in retaining walls and water-impounding structures, is seldom as troublesome, from the standpoint of susceptibility to frost deterioration, as the made joint, possibly because it offers more resistance to the passage of moisture. On the other hand, in pavements and roads, cracks probably cause as much or more frost deterioration than do joints. Besides the common shrinkage crack, there are others that are due to faults of design such as those caused by settlement or other structural movements and insufficient coverage of embedded steel shapes or reinforcement. Given a source of moisture, the concrete adjacent to cracks will deteriorate similarly to that along joints and from the same causes.

A great deal of frost deterioration is due to lack of provision for the drainage of water that falls on or is collected by a structure in the course of its service. Examples of this type of trouble are everywhere. They may be observed in such places as in arch bridges at the junction of the ribs and piers, on surfaces where the drip from drains impinges or runs down, at the exit of horizontal drains, in ballasted decks of railway bridges, in hollow piers and abutments where the drainage system provided is inadequate or has become inoperative, and behind parapet walls.

Another very common fault, sometimes of design and sometimes of workmanship, is a lack of sufficient slope on horizontal surfaces such as sidewalks, the tops of deck slabs, bridge seats, walls, piers, etc., where little or no provision has been made for draining off the water. No workman, regardless of his skill, can finish a large horizontal surface truly plane; he will leave some depressions, however slight, in which water can collect and stand. It has been the experience of the writer that wherever such spots occur, scaling develops even with concrete otherwise resistant to frost action. In fact, one is able to predict the location and extent of future scaling from the appearance of horizontal surfaces after being wet.

Other things being equal, frost deterioration will depend on the severity of the exposure to which the concrete is subjected. The severity of exposure can be resolved into two components; one, the degree of saturation to which a concrete is liable when freezing occurs, and two, the probable number of times during the year that the temperature of its exposed faces is likely to drop below 32 deg. F. A complete measure of the severity of an exposure is the frequency of the simultaneous occurrence of these two conditions.

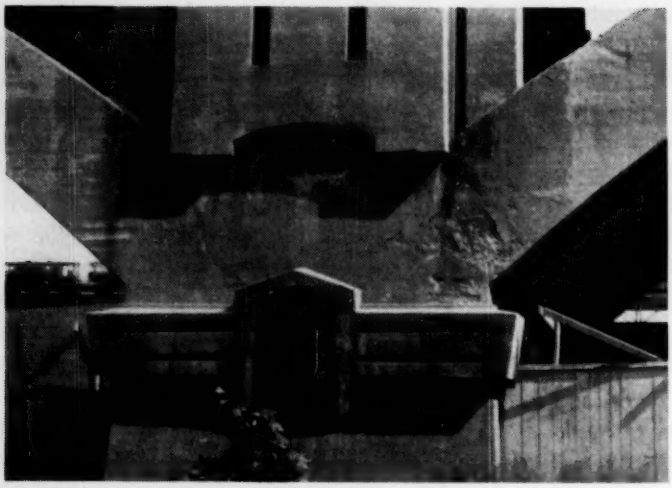

FIG. 8—DETERIORATION OF CONCRETE CAUSED BY DRAINAGE CAUGHT AT THE JUNCTION OF ARCH RIB, AND PIER.

The liability to saturation depends on the position of the concrete. A sloping surface such as the downstream face of a gravity dam, will generally retain moisture for longer periods than a vertical surface, and a horizontal surface longer than either. This is especially true where the moisture occurs initially in the form of snow or ice. Angles and pockets remain saturated for longer periods than those parts of the same structure where moisture drains away quickly. Saturation is continuous where the concrete is in direct contact with a source of moisture such as at the water line of bridge piers and dams or at the ground line of foundations and walls.

The frequency with which the temperature passes downward through the freezing point of water, i. e., on the number of cycles of freezing and thawing, is a matter of climate and location. Thus, frost deterioration is non-existent in the tropics and sub-tropics except at the higher elevations. As one proceeds northward, frost deterioration will be found even in districts of only occasional frosts, as in the midsouth. In the United States and Canada, frost action is more prevalent and severe on the south and west sides of structures than on the north and east, again a function of the number of cycles of freezing and thawing. It is common experience to find concrete of indifferent quality in first class condition at the water line of a dam on a river flowing south, i. e. the north side, while the downstream or south face is noticeably deteriorated; the reverse will be true on

a river flowing north. Some idea of the probable frequency of freezing cycles in any locality may be obtained from a study of the daily mean temperatures compiled by meteorologists. However, these are based on temperatures taken in the shade whereas, in sunny weather, the exposed surface of concrete may pass one or more times through the freezing point on a day that is recorded as continuously below 32 deg. F.

Up to now, this paper has dealt entirely with the causes for the deterioration of concrete from frost action but this is not its primary purpose, which is to indicate how frost resistant concrete may be made. However, the former is necessary to a determination of the latter, just as in medicine, diagnosis is necessary before medication. But in medicine, medication is for the purpose of curing sickness, whereas this paper deals with prevention rather than cure, for the subject of the repair of concrete already damaged, has been dealt with at some length by the writer and others in the publications of the Institute, to which those having problems of rehabilitation are, referred.

A frost resistant structure may be said to start with its designer who should, at all times, keep in mind, the question of its exposure and maintenance. For example, he should provide sufficient drains wherever needed, so designed that they will not plug or otherwise become inoperative in use. Pockets where moisture can collect and stand should be guarded against, such as behind parapet walls and where intersecting surfaces form sharp angles. Horizontal surfaces should be provided with sufficient fall so that the inevitable inequalities of finishing will not leave low areas from which water will not drain. If at all avoidable, drains should not be allowed to spill onto concrete surfaces but should be extended so that the drainage will fall free. Other precautions will suggest themselves to the alert designer.

Construction joints should also receive careful consideration. Those that have to perform as contraction joints should be so designed that when fully closed, destructive pressures that might weaken the concrete by overstressing, will not be developed for this makes them more liable to frost attack. Chamfering the edges of joints to avoid pinching is a good way of preventing this sort of trouble. Joints, the purpose of which is solely to divide the structure into convenient units for construction, should be carefully laid out in advance, not left to the whim of the builder. Horizontal joints should be avoided near the ground line of foundations or near the water line of water im-

pounding structures. One way of solving the horizontal joint problem is to eliminate them wherever possible. The organization with which the writer is connected, has done this since 1930 in all retaining walls, gravity dams, sluiceway piers, etc., up to 70 feet in height, casting these in one continuous operation. No difficulty that could not be anticipated by careful planning has been experienced with this method. From the standpoint of appearance and maintenance, the practice has been a complete success.

A primary requisite for frost resistant concrete is impermeability and low absorption, and the technical literature has, for years, contained many excellent articles describing how such concrete may be made. Most authorities recommend for concrete exposed to weathering, a minimum 28-day compressive strength of at least 3000 p.s.i. and a maximum water-cement ratio of not over 6 to 6½ U. S. gallons of water per sack of cement. In general, these requirements will result in frost resistant concrete, provided materials and workmanship are satisfactory and while durable concrete may be obtained with lower strengths or more mixing water, any lessening of these standards should be done only after consideration of both the particular materials to be used and the exposure to which the structure will be subjected.

Since the characteristics of the concrete mixture are so important a factor in obtaining frost resistant concrete, care should be taken to see that there is plenty of mortar present to carry the coarse aggregate and any error should be on the side of oversanding. The sand should contain plenty of fines, at least ten percent passing the No. 50 sieve, preferably more. If, for water-tight concrete, a choice has to be made between a sand coarser than this and one approaching the upper limit of fineness allowed by standard specifications, that is, 30 percent passing the No. 50 sieve, the writer would favor the latter, in spite of the fact that more cement would be required. Many will not agree with these particular recommendations, and, the writer, knowing this, makes them with some reluctance but he is of the firm opinion, based on no little experience and observation, that they are sound and that much concrete which would otherwise be satisfactory, has scaled and deteriorated because the fine aggregate used in its construction was lacking in essential fines.

It should be remembered also, that some of the fines necessary for good concrete are supplied by the cement. While it may seem to be good economy to use lean mixes, it should be done cautiously and with full knowledge of all the circumstances, for, depending on the

size and type of aggregates, and the method of placing, a certain minimum amount of cement is required for purposes of density. It is true that we are learning how to make better concretes with less cement but for the average job it is still a safe rule to avoid lean mixes.

Workmanship is of the first importance in obtaining a frost resistant concrete. Every means should be taken to avoid segregation and all handling and placing equipment should be so chosen and arranged as to avoid, as far as possible, separation of the mortar from the aggregates. Chutes, if permitted at all, should be confined to short lengths for final placing; the handling of concrete in the forms should be kept to a minimum, i. e., it should be placed initially in that part of the form it is intended finally to occupy; vibrators should not be used to move concrete from one part of the form to another; overworking of vertical faces should be guarded against and only enough puddling done to keep the large aggregate away from the forms. Over-finishing of horizontal surfaces should be avoided, especially where they are to be exposed to the weather; a float finish is, on the average, more resistant to frost than one that has been steel trowelled, not because of anything inherently wrong with the latter but because less skill is required to produce it satisfactorily under average job conditions.

Although not discussed at any length when considering defects commonly giving rise to frost deterioration, the use of excess water is frequently a contributory cause of segregation, particularly to that form known as "water gain," in which water tends to separate from the mix and collect in the top part of sections being cast. Thus the use of overwet mixes is to be avoided as increasing the likelihood of segregation and its attendant evils and in planning for frost resistant concrete, care should be taken that the means for handling and placing the mixes do not jeopardize its quality by requiring wet consistencies for their successful and economical use.

Finally, skilled and adequate supervision is among the requirements for frost resistant concrete. It should take the form of attention to all of the usual factors considered necessary to good concrete, with particular attention to those features that have been indicated here as being the principal causes for frost deterioration. The supervision will usually be performed by inspectors under the direction of an engineer and it is important that these inspectors be experienced men of good judgment, cognizant of the importance of attention to details. Few good jobs just happen; they will be found to have had expert and

detailed supervision, usually by a resident inspector and anyone desiring frost resistant concrete should not overlook this important and highly significant fact.

In conclusion, let me repeat that the requirements for frost resistant concrete are only those already accepted as necessary for quality concrete. However, experience has indicated that more than average care should be taken to have it impermeable and of low porosity and absorption, especially in those places where the structure may be wet for considerable periods of time. Harsh mixes, coarse sands, excess water and segregation from whatever cause, should be avoided. Finally, concreting should be done only under competent and adequate supervision. These are in brief, the requirements for frost resistant concrete.

Discussion to close in Supplement *to be issued with September 1940* JOURNAL, *should reach A. C. I. Secretary by June 1.*

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *SEPTEMBER 1940*

Discussion of a paper by R. B. Young:

Frost Resistant Concrete*

BY L. W. WALTER, R. B. YOUNG, BENJAMIN WILK,
JOHN R. NICHOLS AND W. F. WAY

CONVENTION DISCUSSION

L. W. Walter†—In view of Mr. Young's broad experience with concrete (and here I am going outside the scope of his subject) I should like to have, and have the rest of you have, the advantage of his opinion: If you were designing concrete—placing concrete to protect steel against corrosion in marine structures, salt water and air, would you use any different type of concrete for the protection of steel than you would for protection from freezing and extreme cold? Would one serve the purpose of both?

Mr. Young—I don't know the answer. I have seen concrete up and down the coast in which there were instances of very severe pressure on the concrete brought about by rusting of steel. In almost all cases the concrete seemed to be below normal in quality, and our experience in salt air areas would indicate that concrete resistant to frost will also withstand spalling and rusting steel, because the steel does not get a chance to rust. But I have seen structures in the far south, reputedly well made (well made as far as observation went, several years after construction) in which there were indications of cracking over the steel in concrete that must have tested 3000 to 4000 p.s.i., so I do not know what you would have to do in such regions to provide against such contingencies.

Benjamin Wilk‡—A question arises in reference to 3000 lb. concrete at 28 days. What modification of mix or procedure would be neces-

*JOURNAL, Amer. Concrete Inst., Apr. 1940.
†Erie Railroad, Jersey City, N. J.
‡Detroit, Mich.

sary when October and November low temperature came along? How do you still provide 3000 lb. concrete?

Mr. Young—No change is required in the mix for winter work. The problem in winter concreting is one of curing and protection and its solution depends on the type of structure being built. For any structural frame, the concrete should be cured for strength, the governing factor being that particular strength required to permit it to carry its own dead load and whatever live loads are present during the construction period. If the concrete is to be exposed to weather as in a dam, then curing should be planned to develop sufficient durability in its exposed surfaces to resist frost action. In some structures, such as bridges, both types of curing are required.

John R. Nichols†—I understood Mr. Young to say that to avoid segregation he would avoid harsh concrete and go in for workability. I would expect that a high degree of workability would be secured by adding sand to concrete.

Mr. Young—Yes, but not to an excessive degree. I consider over-sanding much safer than under-sanding, and excessive over-sanding can be avoided if it is kept below the point where appreciably more cement is required to maintain the proper water-cement relations at the required consistency.

Mr. Nichols—Well, I think it does, but there is this, that harshness on one side may lead to porosity and possible segregation; with very high workability, on the other side, too much richness, too much water, too much cement, I would not think you would require sand. Haven't you got to have a concrete for durability that will be just about right?

Mr. Young—Well, on occasion, in years gone by when it was a little harder to get our people to insist on separate aggregates, we had to use pit-run aggregates that ran as high as sixty per cent sand; we put those in and have no experience that would indicate that they were less durable than those that had a more desirable grading. They naturally would have more sand and would require more cement to get the same strength.

BY W. F. WAY*

I have read and re-read Mr. Young's paper several times, and the numerous underscored statements in my JOURNAL vouch for my

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*Vice-Pres., Stuart Cameron & Co. Ltd., Contractors, Vancouver, B. C.

hearty approval. The paper is remarkable, alike for what it says and also for what it does not say.

Mr. Young's is not the first diagnosis of the major ills which beset poor concrete, nor are the simple remedies by which they may be overcome, presented in his paper for the first time. But a good sane restatement of the fundamental verities is timely.

It constitutes a clear, comprehensive statement of the typical troubles and weaknesses that attack porous concrete in *any severe climate*, be it in the Rockies, the Sierras, or Down East. This paper in fact, amounts to an almost complete tabulation of concrete's major defects—only unsoundness is lacking from the list, and that is due to quite other causes than those normally involved in frost resistance. When the enumerated faults are rectified, concrete becomes a thoroughly satisfactory weather-resistant material, whereas if not corrected, it fails signally to render the service to be expected of it.

The paper draws its conclusions from the behaviour of concrete structures according to their *actual behaviour* in contrast to predicting behaviour from laboratory tests which is now so prevalent.

As for what Mr. Young does not say: attention may well be directed to the fact that he fails utterly to correlate excessive porosity, segregation, over-watered mixtures, under-sanded mixtures, construction joints, lack of drainage, adequate supervision—these fundamental weaknesses of concrete in structures—he fails to correlate them with dicalcium silicate, puzzolanos, heat of hydration, sugar test, dry clinker, or any of the current wonders and marvels now being studied in the research laboratories. After a dozen years of dither about cement composition, Mr. Young draws his conclusions by ignoring completely the findings of the cement composition researcher.

Therefore it is suggested that someone—not necessarily Mr. Young —correlate the percentage of tri-calcium aluminate and some of these other things with the behaviour of concrete in structures. Suffice it for the present to suggest that the thanks of the entire membership of the Institute are due Mr. Young for a paper outstanding in worth, horse-sense and just good plain gumption.

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Tolerances in Building Construction*

By John R. Nichols†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

Tolerances are proposed tentatively, allowable variation from the exactly plumb, straight, level and true, for lines, levels and dimensions of reinforced concrete buildings and it is suggested that eventually, after discussion, a set of some such tolerances be adopted as standard by the American Concrete Institute.

Whether the specifications call for it explicitly or not, there is implied in every set of plans that columns, walls and arrises shall be plumb, that floors, ceilings, lintels shall be level—or shall have the slopes indicated, in plane or other geometrical form. It is expected that the structure and its parts shall correspond with the dimensions given. As a matter of fact no building is ever plumb, level, straight and true to dimension,—that is, not *exactly*. Fortunately no building need be. Most buildings are nearly enough to exact size and line to perform safely and satisfactorily their several functions. But occasionally a concrete structure is stripped of its forms to display lines that were not intended, and that are not satisfactory to the engineer or architect, or to the owner. Then may arise the question how far a structure may depart from exact line and dimension and still be considered to accord sufficiently with the plans. How much variation is consistent with full performance of the contract? This is a matter of "tolerance."

It seems to me that it would be useful to the engineering profession and to the construction industry to have established a standard set

*Presented 36th Annual Convention American Concrete Institute, Chicago, Feb. 27-29, 1940.
†Consulting Engineer, Boston.

of tolerances for reinforced concrete structures. They need not invariably be enforced where the nature or use of the structure obviously does not call for the standard degree of accuracy. On the other hand, the existence of standard tolerances need not preclude the specification of finer accuracy where the designer considers the additional expense involved to be justified. In the absence of specifications to the contrary, standard and generally accepted tolerances would serve to limit the variation from exact line and dimension which ought to be and would have to be accepted in any particular case as performance of the contract.

If builders come to confine their inadvertent variations in construction within a set of standard tolerances, and thus bring under control a widely roaming variable now presumably covered by the factor of safety, designers can base their computations upon anticipated performance and avoid being faced with unexpected adverse variations which may seem to threaten the stability of their structures. At any rate the designer will know in advance for how much variation he is charged with the duty to provide.

The suggestions which I shall offer are concerned with reinforced concrete structures. The steel companies have devised tolerances for the manufacture and fabrication of structural steel, but none are provided, so far as I know, for its erection. Structural steel building frames are notorious for their tendency to go out of plumb. We are accustomed to seeing them restrained with heavy guys and turnbuckles during construction in the hope that the walls and floor systems will somehow control them in later life. It would perhaps be unreasonable to impose upon a steel building frame tolerances which will seem liberal and easy in concrete construction.

Tolerances must be related on the one hand to their reasonableness, to the cost of building within them; and on the other hand to the need for, and the value of, close adherence to indicated line and grade. In judging any proposed tolerance, therefore, one must inquire first, "Is it necessary and is it sufficient to build within this tolerance in order that the structure may have suitable appearance, may satisfy the purpose for which it is erected, may be structurally safe and may accommodate the tiling, plaster or other materials that may be added in finishing it?"; second, "Can such accuracy be obtained reasonably, that is, without unjustified cost?"

The elevator manufacturers have for some years asked that hatchways be guaranteed to be plumb within a tolerance of one inch. So far as I know, builders have never hesitated to give this guarantee and for the most part have found no difficulty in building within

that tolerance. It is believed that the tolerances proposed in the following table are in line with this precedent.

TOLERANCES FOR REINFORCED CONCRETE BUILDING CONSTRUCTION

(These tolerances are maxima for individual cases. Strict accuracy should be aimed at and work should be regarded as unsatisfactory if a considerable proportion of it violates tolerances of one half the amounts set down in this table.)

1. Variation from the vertical in the lines and surfaces of columns, piers, walls, and in arrises in general:	in any story ..... ½ inch in any 50 feet or more ..... 1 inch
For exposed corner columns and other conspicuous lines:	one half these amounts.
2. Variation from the level in floors, ceilings and beam soffits (or from the grades indicated on the plans):	in any ten feet ..... ¼ inch in any bay ..... ½ inch in 40 feet or more ..... 1 inch
For exposed exterior lintels, sills and such conspicuous lines:	one half these amounts.
3. Variations from the straight, or from correct position in plan, of building lines; position of columns and walls; dimensions:	in any bay or 20 feet ..... ½ inch in 40 feet or more ..... 1 inch
4. The size and location of sleeves, pits, floor openings, etc.; the location of bolts, inserts and fastenings:	..... ½ inch
For the location of anchor bolts for structural steel, machinery and the like, unless provided with sleeves or other means for adjustment:	..... ¼ inch
Spacing of such bolts in a group:	..... ⅛ inch
5. Variation in cross-sectional dimensions of columns and beams; thickness of slabs and walls:	minus ..... ¼ inch plus ..... ½ inch
6. Footings (a) Dimensions	minus ..... ½ inch plus ..... 2 inches
(b) Misplacement or eccentricity:	2 per cent of the footing width in the direction of the misplacement but not more than ..... 2 inches
7. Variation in the steps of a flight of stairs:	rise ..... ¼ inch tread ..... ½ inch
but in consecutive steps:	rise ..... ⅛ inch tread ..... ¼ inch

8. Variation from dimension in the fabrication of:	
(a) stirrups, column ties and spirals:	.....¼ inch
(b) other bars:	.....½ inch
9. Placement of reinforcement affecting protective covering or effective depth in bending:	
(a) in slabs and in members not over one foot in transverse dimension (in the direction of the tolerance):	.....¼ inch
(b) in other members:	.....½ inch
10. Spacing of bars indicated to be evenly spaced in a group,	
(a) variation from even spacing:	.....2 inches
(b) variation in average spacing affecting the number of bars in the group:	.....5 per cent
(c) minimum clearance between parallel bars:	.....¼ inch

The appearance on plans of specific tolerances in connection with any dimension, or the words "not more" or "not less," would supersede the standard tolerances. Nor may it be thought that any agreement on general tolerances could ever sanction the encroachment of a building beyond the line of its lot or other legal boundary.

Tolerances in the strength of materials are perhaps adequately covered in the specifications of the American Society for Testing Materials. At any rate, I shall make no attempt to discuss them here.

Discussion to close in Supplement *to be issued with September 1940* JOURNAL, *should reach A. C. I. Secretary by June 1.*

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *SEPTEMBER 1940*

Discussion of a paper by John R. Nichols:

Tolerances in Building Construction*

BY JOSEPH DISTASIO, JOHN R. NICHOLS, CHARLES T. KENNEDY, GREER A. ALLEN, GEORGE A. GEIB, WALTER H. WHEELER, A. W. STEPHENS, A. BURTON COHEN

CONVENTION DISCUSSION

Joseph DiStasio†—I find in my experience that we can place bars within a quarter or an eighth of an inch, but when we stop to place concrete and men start stepping all over the bars, bend them all out of place, what tolerance would Mr. Nichols permit there?

Mr. Nichols—The only matter in which the owner, or the engineer representing the owner, is interested is the final location of the bar. I should suppose that any set of tolerances would be relative to that. Where the bar might be at an intermediate stage would be a matter of indifference.

Mr. DiStasio—When you place concrete, it displaces the bars.

Mr. Nichols—Whether one can or cannot build within these tolerances, I suppose there will be all sorts of differences of opinion. If you cannot keep the bars in a slab from rising more than ¼ in. so that the effective depth may be reduced more than that, then you must consider what you will use for effective depth in your design computations. The agreed tolerance should not allow greater than acceptable impairment of the factor of safety.

Mr. DiStasio—I find from my experience that they can keep these tolerances in placing steel except that when they start weaving the bars, in placing concrete they throw them all over the place.

Charles T. Kennedy‡—I think the difficulty might be resolved if Mr. Nichols told us whether that tolerance is in the vertical plane

*JOURNAL, Amer. Concrete Inst., Apr. 1940
†Joseph DiStasio & Co., New York.
‡Consulting Engineer, Cincinnati.

or in the horizontal displacement of the bar. I think that is the question at issue in this particular discussion.

Mr. Nichols—The tolerance is concerned with placement of reinforcement affecting protective covering or effective depth in bending. That is, in the direction of thickness of the slab or perpendicular to the surface of a protective covering. The figure for a slab or thin member (that is, not over a foot) is a quarter of an inch; and in other members a half inch. At least that is what is proposed. Variations in the spacing of bars may properly be much wider than that.

Mr. Nichols—(in response to the question: Is it proposed to design the structure according to the tolerances and not according to theoretical dimensions?) I would propose that the designer design his structure with the tolerances in mind. If he thinks that variation within the tolerances will not too greatly impair the margin of safety, he will not add anything to his design dimensions when he marks them on the plan. But if he thinks his design admits of no impairment of the margin of safety, he must add the tolerance to his design dimensions. It is up to the designer.

*Greer A. Allen**—I think he should add that quarter inch if he is of the opinion that he is not going to be able to keep the bars in their theoretical position, because a half inch on the theoretical depth or the effective depth of the bar in a six or seven inch slab, would make considerable difference in the stresses of the steel. I do not see why the bars cannot be kept to their theoretical positions. We do not have any trouble in keeping bars in their position when they are occasionally knocked out; they often get them back before we pour the concrete.

George A. Geib†—From the standpoint of a man in the field, I sometimes ask myself what the designer is thinking of. I wonder if designers consider the many factors in building construction in these days of high competition and doubtful labor and the many difficulties in placing bars over a column in a cluster; four or five bars over the top and four or five coming in from the other direction, hitting the upright rods. We are not thinking of tolerances, we are thinking of getting those bars through. Another thing is on slabs, beams and columns; the human element, as we have it today, with doubtful craftsmen; are you going down there with your ruler or micrometer at the bottom of a column to see whether those bars are an inch and a half from the face of your finished column? You have to take what you get and trust to God that it is all right.

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†Washington, D. C.

Mr. Allen—That does not place our engineers on a very sound basis. We should have proper inspection; we should have a man on the job who knows that bars should be in their theoretical positions and he should see that they are in those positions. My idea of trusting to God is that it is not good engineering practice.

Mr. Geib—Take the little matter of runways and other elements that enter into the construction process; you may have done everything you could to see that the best engineering practice was followed, but there are many elements in the matter of placing concrete and the handling of forms and runways that disturb things and change it from what it was originally. It must be a practical proposition, a common-sense proposition. When a thing is radically wrong, you have to rely on common-sense rather than engineering exactness; considering the factors that made it wrong in the first place, and whether it would be wise to stop or continue a pour.

*Walter H. Wheeler**—It seems to me very desirable that the profession set up what it considers a standard of reasonable tolerances. That done we have to keep in mind that when it comes to enforcing a schedule of tolerances, we can enforce only what a court will consider is reasonable compliance with the contract.

A. W. Stephens†—I wish to call attention to another phase of this concrete business. I am heartily in favor of the adoption of some basis for tolerances. I believe it is proper to give thought to what those tolerances will be. Another phase is: what determines the live load on the floor? A discussion probably in the architect's office; one of the associates says "I think we'd better design this floor for 125 lb." Another says "Oh no, it is likely to have a much heavier load, 200 lb. is little enough." The result of the discussion in the architect's office is a compromise on 125 or 175 lb. Possibly the discussion is within 25 or 50 per cent of being right when the building is loaded. Another point: the concrete is designed for 3000 p.s.i. There will be a variation from that three thousand pounds—more or even less than three thousand. There is another factor, the stress capacity of the rods; there probably is some variation in the yield point. It would seem to me, then, that we can with all safety adopt reasonable tolerances and design our members on the basis of theoretical depths. I think that we can do that safely, and having due consideration for the premises in the problem, that is, the variation that may occur in live loads, the concrete as such, possibly the rein-

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†Washington, D. C.

forcing steel, and I am heartily in favor of using the designer's actual theoretical depth for design and to adopt reasonable tolerances therewith.

*A. Burton Cohen**—Why don't we hide those tolerances in the design? If the code says a 100 lb., design for 105 lb.; don't give the constructor the idea that he has that tolerance. I don't mean to be severe on the contractor, but if you give him a little leeway, he may take some more.

Mr. Nichols—In the very beginning, I pointed out that there was no need to enforce tolerances except when greater variation from the plans involved loss of some valuable quality in the structure. Many phases of building construction do not need the application of tolerances at all. Perhaps the matter of strength and stability of the structure least needs the application of tolerances. In that I have particular regard to what Mr. Stephens and Mr. Cohen said. But I think there is a real need for tolerances in the appearance of the exposed part of a concrete building. I have seen concrete buildings which, when the forms were stripped, were much unlike what they were intended and ought to look like. The question then arises, must the contractor correct the error or is the result near enough according to plans? How much can a lintel sag and still be level? How much may it be tipped to one side and still be level? How wavy can a floor and ceiling be and still be level? It seems to me worth while determining reasonable limits for such variations. And while we are about it let us cover the field by putting down what seem to be reasonable tolerances for the bending and placement of steel, and for those dimensions that have to do with the protection of steel, and the strength of the building. Let us, by all means, be reasonable in this and set up outside limits that can easily be maintained. I have no notion that the figures I have suggested will be accepted by a committee of this Institute. Such a thing never happened before. There is no reason why it should happen. But I think it well that some committee of the Institute should determine what tolerances are appropriate, and set them up as a standard. That is the suggestion I want to make.

*Consulting Engineer, New York.

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The Contribution of Ready-Mixed Concrete to the Building Industry*

By H. F. Thomson†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

The development of commercial service in delivering ready-mixed concrete in urban centers has represented one of the significant advances in building practice during the last fifteen years. The industry has pioneered in introducing more accurate control methods; these refinements now enable small jobs to secure as reliable control of the concrete used as is available on very large jobs. The advances in quality of ready-mixed concrete have influenced improvements in the production of all other concrete used in a community. Ready-mixed concrete offers to the building trade a service which has proven helpful in convenience, speed, ability to handle winter work, and in other respects. The way toward further improvements in concreting practices has been pointed out by this infant in the building field.

The product, ready-mixed concrete, and the service in delivering it have become such accepted factors in the building industry in most American cities, small as well as large, that we may fail to recall how this new service has grown during the last 15 years, or wherein it has contributed to the progress of urban building. It is therefore fitting for us to take stock of the accomplishments of this infant among the fold of concrete practices.

The term Ready-Mixed Concrete is used in the following remarks to describe "mixed concrete delivered to the work ready for use;" which definition is included in the A. S. T. M. standard specification for Ready-Mixed Concrete (Serial Designation C94-38). And in general, the commercial type of operation is referred to as the agency in

*Presented 36th Annual Convention, American Concrete Institute, Chicago, Feb. 27-29, 1940.
†Vice-President, General Material Co., St. Louis, Mo.

position to serve the urban building industry. Fundamentally, the ready-mixed concrete idea applies the modern conception of mass manufacture and centralized control to the production of concrete for scattered building jobs.

A summary of the recently-current conditions regarding materials for and methods of, mixing concrete for city building work, and some of the improvements which followed the introduction of ready-mixed concrete, was given in an article‡ which appeared some years ago, written by the engineer in charge of concrete for municipal construction in a well-regulated mid-western city. The following excerpts from this article appear interesting today:

Previous to 1927 concrete had been generally produced in a rather haphazard manner, inaccurate measurement of aggregates, some cement and plenty of water, having been the rule. . . . The writer was placed in charge (of concrete production) during 1928, and found that although much improvement had been made in the preceeding year, much was left to be done. Concrete was produced by mixers on the jobs, except for one central-mix plant. Commercial wet-hauling of concrete was in its infancy. Scientific production and control of concrete was comparatively new to the department's engineers. . . . For fine aggregate almost any gradation of sand was accepted. . . . Any available quality of limestone was used. . . . On account of the great variations in grading between shipments of aggregate, continual sieve tests were run at the central-mix plant's laboratory—the only field laboratory in the city—and the proportion of fine to coarse aggregate was adjusted from time to time. . . . At the jobs where the mixers were located on the work, the mixer tanks were calibrated and set at the season's start, and the proportion of sand to gravel rarely if ever changed, and the concrete in general took care of itself. The contractor was sure to see that enough water was used. . . . The use of ordinary beam scales was often the cause of overweighing. At the central plant the mixing process was excellent. However, the mixers at the job were very poorly operated; the mixer operators were the judges of the amount of water used, and it was the exception when consecutive batches were of the same consistency. Locking devices on the discharge chutes always seemed to be out of order, so the 30-second mix was common. . . .

In the succeeding years great progress was made in remedying the aforementioned conditions. An educational campaign was started among the material producers and contractors, in which the officials of the central-mix plant gave excellent cooperation. . . . The same cement contents were used in the mixes as before. Gradings of the specified classes of aggregates became much more uniform. Moisture tests were accurately made, and as much care in all particulars was taken in "dry-batch" work as at the wet-mix plant. . . . In measuring quantities of materials, dial scales (preferably springless) were required for all aggregates. On the job-mixers, old water tanks were gradually eliminated and modern ones with automatic control substituted. Locking devices were required to be in working order.

The article then cites the table below as giving the average strengths of cores drilled from pavements laid each year.

‡"Improving Concrete for Public Work" by Chas. W. Barnes, Jr., *Concrete* January, 1935.

Year	6 Sack per cu. yd. Concrete		4 Sack per cu. yd. Concrete	
	No. of Breaks Obtained	Average 28-day Strength, p.s.i.	No. of Breaks Obtained	Average 28-day Strength, p.s.i.
1928	58	2,990	..	.....
1929	48	3,185	..	.....
1930	156	4,056	71	2,661
1931	269	4,329	55	3,166
1932	62	4,725	45	3,182
1933	38	5,236	36	3,580

The comments in this article are significant, as similar improvements in connection with building concrete have occurred in many cities during recent years. A contributor to the increase from year to year in average strength was undoubtedly the improvement in quality of portland cement. But in addition, the introduction of control methods in producing ready-mix gave such definite improvements in strength and quality of the concrete so produced, that operators handling job-mixing endeavored to improve their practices so as to catch up and remain competitive; with the obvious result that the average of concrete production practices in a community were vastly improved. The influence of ready-mix practices in any community extended to job-mixing in that community, and thereby was beneficial in advancing notably the average quality of concrete entering the local building work.

The early operators of ready-mixed concrete in the larger cities were imbued with the pioneering spirit of looking for better ways of producing and serving concrete. The stories of leading operators in various cities some 10 to 15 years ago, were much the same. Many initially installed inundation as being the best control method then known; but although it represented a forward step toward more accuracy in measuring aggregates, this method was found too inflexible for the variety of proportions and consistencies required of a commercial plant. In some plants the "regulator" method was tried, wherein the cement and water were measured in separate boxes supported on opposite ends of a beam, the position of the fulcrum supporting the beam being moved for change in water-cement ratio. Also, the so-called "hydro-graphic" method (which was a combination of weight and inundation of the fine aggregate) was tried. But each of these was in turn discarded in favor of straight-weighing of both aggregates on scales having suitable accuracy, which has since become the almost universally required method for job-mixing on supervised work of any importance. It is notable that the weighing of aggregates was early proven practical in ready-mix plants for use in connection with urban construction. These changes were expen-

sive for any ready-mix operator, sometimes involving the junking of good equipment, but were made for the purpose of better control.

And the same occurred with delivery equipment. The ordinary square-cornered dump-body gave way to the "fish-tail" and then to the "bath-tub" body with its chamfered corners. Then, as a demand for truck-mixing or agitation appeared, the early units with revolving paddles of various arrangements were succeeded by revolving drums with relatively small diameters, and later by drums with large diameters as compared to lengths. Such changes in a few years, long before equipment was worn out, were costly to the operators previously producing ready-mix, but were made in the interest of offering the building trade, a better concrete with more convenient delivery arrangements. Such pioneering spirit has contributed to the rapid introduction of ready-mix as an accepted method upon urban building work. As regards currently used types of equipment, it should be noted that in addition to revolving cylinderical drums as just mentioned, certain special designs of revolving cones and of revolving blades are being operated successfully.

The development of the ready-mix industry in recent years has departed in two important features from what was anticipated by many of the earlier operators. In the first place, these operators invested in improved control and delivery equipment with the expectation that they could capitalize on the better quality of their product and on their specialized knowledge of how to produce a given quality more economically than could be done under ordinary job conditions. However, the insistence of purchasing and supervising agencies upon specifying how the concrete is to be produced as well as the quality desired—for instance naming the cement content as well as the minimum strength—has removed much of the incentive to furnish high quality concrete economically. And in the second place, there is so much of a public-utility-service character about a ready-mix operation that cold logic from the standpoint of ability-to-serve all demands of the local trade, points toward the desirability of a single strong, well-equipped operation in an urban area, just as experience has evolved a single telephone system or a single electric distribution in most of the larger cities. For a given amount of ready-mix equipment in a locality, better service would be assured to builders if that equipment were in the hands of a single operator, instead of being distributed among several operators, which has been the recent trend as resulting from uncontrolled competitive forces. The result of both

of these tendencies has been a distinct check upon the pioneering and progressive spirit which imbued the early operators.

Within recent years the production of concrete under acceptable supervision has become a manufacturing operation in lieu of the former rather haphazard mixing. The large concrete plants which have been set up for the important construction projects such as power, navigation, or irrigation dams, are notable examples of the application of control methods, as well as large volume production. The present control methods in such plants were developed concurrently and along similar lines when the relative rates of production are considered, as the methods developed and adopted by commercial ready-mixed concrete producers. Thus the responsible ready-mix plants are now offering to the urban building industry the same type of effective control which is used on very large, centralized projects. There is nothing in such control which the builder could not introduce in job-mixing if he undertook to give the attention to detail and the degree of expert supervision with which both the large projects and the established commercial producers have surrounded their operations; but the fact is that the individual builder has considered such control as too expensive. In other words, commercial ready-mix now offers to the small job concrete produced under accurate control such as would otherwise be available only to very large jobs.

It is of course recognized that all ready-mix producers may not have lived up to their obligation to furnish a control which is superior to the ordinary job-mixing, just as some may be found in any line who are inclined to operate in the easiest, instead of the best way. In localities where such are located, a neighboring operation which incorporates good control may be expected to merit discriminating support. Nevertheless, it cannot be gainsaid but that the ready-mixed concrete industry as a whole has served to raise the level of the quality of concrete used in building work.

The influence of good control as exemplified by responsible ready-mix operators extends far beyond the scope of such operations themselves. The experience with aggregates is typical. In order to produce uniform concrete, it is necessary that the aggregates be uniform, especially in grading. As uniformity in production is imperative for a ready-mix operator, who is likewise an important factor as a buyer in a local aggregate field, his demand for uniform grading in successive shipments and on successive days, has naturally influenced the aggregate producer to improve his own production methods to maintain uniformity. But as the aggregate producer supplies other trade as

well, the improvements in aggregates are passed on to the entire local trade, with the result that all local concrete benefits accordingly. It would appear difficult to overemphasize the influence for better grading of aggregates which has accompanied the urban development of ready-mixed concrete.

As mentioned previously, one of the developments introduced by the pioneer ready-mix operators of ten to fifteen years ago, was the use of weighing for measuring aggregates. This was an inevitable step in the progress of the industry because no other method of measurement offered the flexibility necessary for rapid handling of successive batches of different sizes and proportions, such as is required in a commercial operation. This contribution toward effective control of uniform production was quickly recognized, with the result that the weighing of aggregates has become the accepted method of accurate measurement of materials on all types of work where good control of concrete quality is the order. And in this connection it may not be amiss to point out that one of the earliest efforts to use automatic weighing in concrete production (in this case, applying to cement measurement) was incorporated in a commercial ready-mix plant; this installation appears to have been the forerunner of the extensive use of automatic weighing upon the major dam projects of recent years.

The accurate measurement of the water added to a batch, and the routine determination of the moisture in the aggregates in order to know accurately the total water present in a batch, were further details of control which were pioneered in connection with ready-mixed concrete. The development of central-syphon tanks for accurate measurement of water, was largely the result of demand from ready-mix operators for a more accurate and reliable means of measuring water than the former practice of expecting the mixer operator to watch the fall of the maniscus in a gage glass and to cut off the flow at exactly the designated point. The accurate determination of water-quantity, largely as a means to greater uniformity, was also in line with the then current attention which was directed to the water-cement ratio as a measure of controlling quality of concrete. The term "water-cement ratio" means nothing in the absence of accurate information upon the quantities of moisture and added water, as in the case with most job-mixing; as reliable information regarding the water-content can be supplied the builder and supervisor by the efficient ready-mix plant, this type of operation

has contributed significantly to modern thought regarding factors for controlling the quality of concrete.

The combined result of these control measures which were adopted early and have become standard in ready-mix operation—namely, use of uniformly graded aggregates, accurate weighing of aggregates and measurement of cement, and reliable measurement of water—has been to produce a uniformity of concrete, with known production data, which now justifies reliance on strength as a basis for specifying the desired quality in the structure. Here again, a strength control can be applied to job-mixing on a large project, such as a major dam; but a reliable ready-mix operation can offer to an ordinary building job, the same method of specifying the minimum quality of concrete required and of insuring that such quality is furnished. The availability of reliable strength-concrete made possible by these developments has given to designers a basis for more accurate, and therefore economical, designing than could be done safely when the somewhat haphazard job-mixing was the only available method for producing concrete. Also, in cases where exceptionally strong concrete is needed for a given design, a well-controlled ready-mix operation may offer the most reliable means of meeting such requirement.

From the standpoint of the contractor as representing the user of concrete, the development of ready-mix has also contributed benefits. It has been demonstrated repeatedly that the concreting proceeds faster when ready-mixed is used (assuming that the producer has adequate delivery equipment available) than when job mixing is relied upon. What ready-mix producer has not had the experience of seeing a customer's forms give way, simply because forms which were strong enough when job-mixing was used, were not adequate to withstand the greater hydrostatic pressure resulting from the faster placing incident to the more rapid delivery of the concrete? Such increased speed of placing results in lower labor cost, and in a desirable general speeding-up of the job. An instance was noted recently wherein a contractor had planned to handle concrete on a multistory building at from 15 to 18 yards per hour; but after a short experience with ready-mix, he stepped up to 30 to 40 yards per hour, as he was able to get such faster service from the commercial producer, than he could have done with his job-mixing equipment. Such performance represents a contribution to the building industry.

An item which should not be overlooked is the distinct assistance rendered to modern winter concreting practice through the ability of the ready-mix producer to offer warm concrete through his central-

ized heating of materials, as compared to the too-frequent former practice of mixing the concrete at any temperature above freezing and then attempting to supply heat externally to the concrete in place, as by salamanders, to induce setting and hardening. Ready-mix production has simplified the use of warm concrete on jobs of any size. It has become an important agent in enabling building work to proceed through the winter.

Various other benefits have been offered to the contractor-consumer by ready-mixed concrete, such as convenience, eliminating noise and dirt, minimizing the contractors employment, facilitating work on crowded sites where space for material piles and mixing plant is not available, releasing the contractor's organization to concentrate on placing, etc.; but to discuss such features in any detail would approach common "sales-talk," which is not the purpose of the present discussion.

An era of more accurate estimating of cost of concrete work, as well as economy in designing, has followed the introduction of controlled ready-mix. When using ready-mix, safe unit prices can be applied to the volume of concrete required for the structure, in lieu of the many uncertainties with job mixing, such as loss of aggregates on the ground, spoilage of cement and disappearance of cement sacks, and further, the addition to the estimate of the final 10 per cent to 20 per cent to cover the various indirect costs. The inherent accuracy of reliable ready-mix production has permitted checks on quantities and values which have been impossible previously in the absence of such accuracy.

This same accuracy of data in connection with ready-mix has also brought into focus other matters pertaining to concreting, which have been unrecognized previously due to lack of adequate information. For instance, the standards for the handling of test specimens were found not sufficiently specific when applied to determine the quality of concrete at early age, say 3 days, as is frequently required of commercial ready-mixed concrete; no standard conditions for temperature or humidity were specified for the time the cylinders were left on the job. This laxity has been corrected in recent standard specifications, which were promulgated after ready-mix producers had pointed out the deficiency; the revised specifications require that specimens be subjected to standard conditions from the time of casting to the arrival at the laboratory.

Many of the problems now facing concrete technicians, such as the question of shrinkage while the concrete is in plastic state, or

change in characteristics during mixing or agitation beyond the minimum, are not problems of ready-mixed concrete solely, but are fundamental problems in connection with all concrete. But these problems were either unrecognized or were accepted blindly as necessary evils, until the improved controls which were introduced by responsible ready-mix operators, have brought such matters to general attention. In the field of investigations, as with other features of present day production of quality concrete, the influence of ready-mix has extended beyond the actual operations involving such concrete itself. The cumulative knowledge regarding concrete has been extended notably by the summaries of information assembled co-operatively through the National Ready Mixed Concrete Association, and also by the researches which have been sponsored by or conducted by individual ready-mix producers.

Today, the concrete from most job mixers on urban construction is not comparable to the product from a reliable ready-mix plant; one is haphazard, the other controlled according to modern standards; and yet the ready-mix quality must frequently compete in price with the other. Further improvement in ready-mix practices is almost discouraged. It is apparent that the degree of control which has been applied by even the more progressive ready-mix producers, does not represent the maximum in control methods. In such a commercial field the extent of refinement of control must be limited to the degree that the purchasing trade will support; and back of the actual purchasers on most important work are the designers and supervisors, the architects and engineers. Recognition that the responsible ready-mix operator offers to the building industry an improved method of producing quality concrete, has been slow in developing, and so far there has been little indication that any further improvements in control which may become available, would bring compensation to cover the added cost. In one notable instance, which is typical of others, the operator installed a highly developed system for controlling the design and quality of each individual batch; but after a year's experience the system had to be abandoned and more routine methods installed, for the reason that such specially-controlled concrete could not command any premium over competitive outputs produced under less complete control. But it is to be expected that as the beneficial possibilities of good control become better appreciated, there will come greater support for further improvements. The progressive ready-mix producer is in position and is the logical agency to provide such further refinements for which compensation may be forthcoming.

To summarize, the development of commercial ready-mix concrete production during recent years may be considered a notable contribution to the building industry in the following comprehensive respects:

1. Pioneering more accurate control methods, which refinements have resulted in better concrete so handled.

2. Influencing improvement in much other concrete production.

3. Offering to the building contractor a service which has proven helpful in convenience, speed and other respects.

4. Pointing the way to further improvements in concreting practices. Accordingly, ready-mix concrete represents one of the most important forward steps in concreting technique of the recent years.

Discussion to close in Supplement *to be issued with September 1940* JOURNAL, *should reach A. C. I. Secretary by June 1.*

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Discussion of a paper by H. F. Thomson:

The Contribution of Ready-Mixed Concrete to the Building Industry*

BY A. S. BROCK, H. F. THOMSON, ALEXANDER FOSTER, JR., L. W. WALTER, FRED HUBBARD

CONVENTION DISCUSSION

A. S. Brock†—I should just like to have information as to what is the average specification met by the ready-mixed concrete plants? A few years ago I recall reading a trade publication in which the only quotation on ready-mixed concrete I could find was for the well-known 1:2:4 and 1:3:5 mixes.

A second question is, in case the specification calls for a water-cement ratio, or strength or cement factor, that is, a concrete control on any one of those bases, what refinement and methods of mix design are used by the ready-mixed concrete plants? I would like to have a general idea of how much opportunity is taken to use the refinements of concrete mix design.

A third question is what attention is given to grading to obtain a sufficient quality of fines?

Mr. Thomson—I think the first question is probably answered very accurately in Mr. Foster's paper (February JOURNAL) and I don't want to steal his thunder. As I recall that paper gave a list of a large number of operations which their plant is qualified to conduct. The company with which I am connected feels qualified to furnish concrete under any specifications, within reasonable limits. That includes proportion mixes, strength, consistency, water-cement ratio and so on. I would not attempt to summarize the variety of mixes called for.

*JOURNAL, Amer. Concrete Inst., Apr. 1940.
†Portland Cement Assn., Chicago.

One item might be of interest. Although bag cement is used at the plant to which I have referred, our operating company has had experience with bulk cement in a number of other plants; and the outcome has been that under the stress of the depression, central mixing plants utilizing bulk cement, have disappeared and the original plant dependent on bag cement has survived. A tabulation a short time ago showed eighteen designations of cement passing through that plant within two weeks. This does not mean necessarily brands of cement, but different tests for different laboratories, high early strength, etc. We have found that handling eighteen designations of cement within a very limited period, introduced mechanical complications which were a factor in forcing us off bulk cement.

The question of grading of aggregates was mentioned. Some years ago our company set up tolerances of gradings of commercial sizes to be met by the aggregate producers. At one time we went further; we attempted to handle separated sizes under somewhat the same setup that the Public Roads Administration found effective on road construction. Our plant was equipped to utilize separated sizes, from which we expected greater uniformity and therefore improved quality of concrete. Even though we had set up tolerances approved first by the producers and later by the city and state authorities, there was sufficient variation in the product of a given size received from different producers so that we could not get the uniformity of aggregates in our stock piles which we thought was necessary, and we finally had to go back to our original graded sizes. I think commercial plants, certainly the larger plants, are in position to exert a definite influence on the commercial producers of aggregates by adhering to and insisting that the successive shipments meet tolerances. In our own case we have at times made rejections which caused expense to our producing friends but which we thought necessary to maintain quality in our product. I think a good many of the larger ready-mix producers have been very careful in their insistence on grading.

Commercial plants are somewhat the victims of local producing circumstances. In our locality the practice in producing sand has led to an unduly coarse sand with a deficiency of finer sizes which are washed away. Two general classifications of sand are available in the market, one for concrete and one for mason's work. The mason's sand runs very much higher in the smaller sizes. So by stocking both sands there is opportunity to blend the finer sand with the coarser sand to give us a degree of fines in our final mix which is superior to that obtained with regular concrete sand.

*Alexander Foster, Jr.**—We are also producers. Our inspectors have the right to reject materials found unsatisfactory. We endeavor to have a minimum of 12 per cent and an average in the vicinity of 15 per cent of the concrete sand passing the 50-mesh sieve. Our tests lead us to believe that a sand having a high percentage passing 50 mesh and a low percentage passing 28 mesh produces good concrete. Our concrete sand usually has a minimum of 2 per cent passing the 100-mesh sieve. We are blessed with a building code which limits the minimum amount of cement which may be used for varying types of work and for varying strength classifications of concrete. We do not endeavor to obtain strength with a minimum amount of cement without regard to a water-cement ratio. In the event that there is a higher percentage of crushed particles in the coarse aggregate or the coarse aggregate contains less fines than usual, our inspectors are authorized to upset the mix by increasing the sand content, decreasing the coarse aggregate by a like amount and adding more cement if that be necessary to obtain the desired workability without violating the water-cement ratio. In the course of a year, however, the amount of added cement for the above reasons is comparatively small. This is mainly due to the close check kept on the grading of our coarse aggregates.

L. W. Walter†—I think there is increasing interest in the problem of temperature control of concrete at the time of placing. I should like to ask what demands are made upon producers as to temperature limitations. In winter the temperature requirements specified for concrete are, I believe, sometimes needlessly high. Are the ready-mixed concrete plant operators confronted with need for better temperature control?

Mr. Foster—We have a table on page 268 showing by a check-up in seventeen tests run recently, the temperatures of concrete as low as 59° and one as high as 70° obtained usually by heating the water. The specific heat of water is sufficiently great to give us a normal temperature. We can run live steam into the tanks containing fine aggregate, but I think Mr. Young has found that steam pipes and not condensed steam, in the fine aggregate, are better. We have a large horse power boiler which keeps the water at high temperature. In days when the air temperature ranges from 17° to 25°F, the average temperature of the concrete was 67° to 68°F.

Mr. Walter—In the use of artificial, light-weight aggregates or slag, to what extent do you find it necessary to pre-wet the material?

*Mr. Foster of Philadelphia is senior author of a paper at the same session.
†Erie Railroad, Jersey City, N. J.

How long in advance of its use and how long a drainage period do you allow in order to maintain a uniform consistency?

Mr. Foster—We have had no experience along that line in recent years. We have, however, conducted a series of tests on light-weight aggregates and have found it very difficult to gauge the slump which might be expected. The coarse aggregate was kept wet in the pile. Most of the mixes, however, resulted in harsh concrete. I have learned that when certain light-weight aggregates have been used it was almost impossible to forecast workability of a mix. We are not so familiar with slag, although during the first few years of the operation of the central-mixed concrete plant slag was used. Most of the concrete was produced for street foundations and the desired slump ran from 1 to 3 in. During the operation of this old central-mixed concrete plant water was not added to the slag until it entered the mixer. In the construction of buildings in the City of Philadelphia where the concrete was mixed at the site of the work and the coarse aggregate used was slag, it was necessary to keep the stock piles of slag damp, particularly during the summer months. There was no specified time of wetting before the slag could be used. Slag was kept wet from its receipt at the site of the work until used. The batches of concrete in the mixer at the site of the work were regulated as to workability by observing the concrete as it came from the mixer; if it was too dry, more water was added. The production of concrete was not a science but an art, and much depended upon the mixer man.

Mr. Thomson—I cannot give a thoroughly satisfactory answer to Mr. Walter's inquiry about light weight aggregate. We have used a good deal, however, and it is a troublesome matter; but through the use of the consistency indicator, we have found that we can control the uniformity of the concrete going to the contractor within satisfactory limits. There are also difficulties in measuring the light-weight aggregate which make trouble for the operator. I think it has been generally established, that the operation through the central mixer plus the slow agitation en route to the job is the equivalent and is superior to the ordinary pre-wetting of the aggregate. By judicious proportioning of fines to coarse and sometimes by the use of an admixture, we believe we have been able to furnish the contractor with a very satisfactory concrete, from the standpoint of workability. Some of the work that has been done in that locality with the light-weight aggregate has a surprisingly satisfactory, uniform texture.

I should like to refer to Mr. Walter's previous question in which he implied the possibility of damage from the overheating of concrete

delivered during cold weather. Naturally an operator believes that heat is money and does not want to put in any more than is absolutely required by his obligation to a particular job. In our case the practice is to deliver the concrete to the job at approximately 70°F. The difficulty that we have found is usually the demand of the job for inordinately high temperatures—thus under a specification which said that the concrete shall reach the job between 70° and 90°; we have had instances of an inspector demanding 90°. Although we believe it is not the best practice, we have at times tried to run up the temperature on specific deliveries as high as 90°. Such a requirement is the result of poor supervision on the part of somebody behind the contractor. In such an instance we would not assume responsibility for the results.

I think it is indicated in Mr. Foster's paper that it is necessary to require heating at temperatures above forty. A contractor who has received heated concrete when the temperature was below 25°, may sometimes still demand heat when the day time temperature is above 40°, so that he may avoid protecting his structure against a fall in temperature during the night. Such a demand is at the expense of the operator, and may indicate lack of proper supervision at the job itself. The ability of the ready-mixed producer to deliver in cold weather is sometimes abused. We have had a severe winter; it happened that one day when the temperature was twelve below zero, we had calls for delivery to certain jobs, which represented substantially forty per cent of normal output in summer. It required extra labor and considerable ingenuity to produce satisfactory concrete under those conditions, because it was unusual to our locality.

*Fred Hubbard**—In reply to Mr. Walter's specific question relative to the pre-wetting of slag, I think it can be said that there are two schools of thought. One is that the slag should be pre-wet before introduction into the mixer, and the other is that the slag be used dry. As you probably know, the production of commercial slag is a dry process all the way through, and the slag ordinarily comes to the mixer dry—to such an extent that it absorbs some of the mixing water which of course has to be compensated for. I might say that by far the most usual procedure is to use the slag dry, in which case the absorption of the aggregate will have to be taken into consideration and more of an allowance made for the difference in slump from the plant to the job, than with either crushed stone or gravel. Where slag is pre-wet, the water is usually added the day or night before the material is used and is applied to the slag while it is in stockpiles, or

*Standard Slag Company, Youngstown, Ohio.

cars, or while being unloaded from cars, so that, when introduced into the mixer, it is as nearly as possible in a saturated, surface-dry condition. Ordinarily there is no trouble in controlling the consistency of slag concrete.

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Resume of Report by Committee 317*

By A. J. Boase
AUTHOR-CHAIRMAN OF THE COMMITTEE

The report of A. C. I. Committee 317, published under the title of "Reinforced Concrete Design Handbook," was sent to the entire membership of the Institute as of December 1939—time of publication.

The committee carefully considered several matters of policy before actual production work was started. It was decided to develop all data on basis of procedures used in standard codes but to avoid taking any stand for or against any code requirement. A related decision was that the report should be prepared in the form of a handbook rather than a textbook. In other words, the subject was to be presented as a time-saving device for the benefit of the experienced designer.

DESIGN, SIMPLE BENDING

The ordinary design formulas for simple bending are written in a form different from the conventional usage, and this is done for several reasons. It is quicker and easier to carry out calculations with short numbers stripped of excess ciphers. Loads are therefore written in kips, and since spans are in feet the transformation from the units of "kips and feet" to "pounds and inches" is accomplished by introducing special coefficients such as a, c, F, and G. These coefficients serve other useful purposes and their use eliminates many tedious computations.

DESIGN, COMBINED BENDING

In regard to design of beam sections subject to either simple bending or combined bending, the handbook calls attention to the striking similarity that exists in the two types of bending. Actually, they are

*A book of 132 6 x 9 pages—a circular on whose further distribution is available from the Institute Secretary on request.

identical problems if a factor, i, is introduced as a part of the product of iA_s. Simple bending is merely the limiting case for i equal to unity, but all formulas are otherwise identical. The eccentricity inserted in the formulas is to be measured from the eccentric load to the tensile reinforcement. The formulas do not apply, however, when the eccentric load is a compressive force that falls between the resultants of tensile and compressive stresses. The case then becomes a problem in column design popularly known as "Case 1."

INVESTIGATION

The procedure presented for determination of stresses may appear to require much calculation work, but its advantages are that it applies to any type of beam problem and gives results with any desired degree of accuracy. It should be noted that the problem in its most general form, illustrated in Example 18, has *ten* variables. To cover this type of problem by means of tabulated or graphic data would require a space out of proportion to the importance of the problem. The procedure chosen is convenient for all types of investigation of rectangular sections, even for the problem in Example 15 which has *eight* variables: compute the steel percentages, select values of k and j from tables and calculate stresses by the usual formulas.

STIRRUPS

The new stirrup chart presented is actually a condensed tabulation of stirrups ready to use for practically all ordinary problems of stirrup spacing. The use of the chart is better understood when it is realized that stirrup locations for any shear triangle or trapezoid with the *same* slope may be laid out on *one single line*. The 36 lines represent 36 slopes. One may use all or any part of each line, as the case may require, by computing reciprocal values of spacing (1/s) at both ends of shear area and by selecting stirrups in that interval only. The stirrup chart is universal in its application in so far as it may be used for any bar size and any allowable stress. Data for five sizes and nine stresses are noted on the chart but other values may be added. In combination with Table 16, the chart may also be used for spacing of stirrups with any inclination between 30 and 90 degrees.

CONCENTRIC LOAD ON COLUMNS

Design data are presented for five codes, for each of which one page is devoted to tied and two to spiralled columns. Whenever it can be done, the elements contributing to load capacity (concrete, longitudinal bars and spirals, if any) are tabulated separately, together with their individual loading capacity. The total capacity can then be

ascertained by addition of its two or three component parts. Occasionally, this general scheme has been modified to meet specific code requirements.

ECCENTRIC LOAD ON COLUMNS

Bending combined with axial load on column sections is treated for moment in one direction only. Six pages of diagrams cover cases of three types of cross-sections in which tension is disregarded. The upper limit in these diagrams corresponds to the case in which the extreme fiber stress is six times the axial load divided by the gross concrete area. Beyond this limit, design rectangular and T-shaped sections as illustrated for beam sections subject to combined bending. Formulas for determination of stresses in uncracked column sections are given below Table 33.

FOOTINGS

Tables are given for design of square spread footings and for footings with from 4 to 20 piles, all designed according to A. C. I. Code, 1936.

APPENDIX

The appendix contains a restatement and an extension of an article published in the JOURNAL entitled "Design Coefficients for Building Frames." New tables and new examples have been added and the explanatory text has been enlarged.

ERRATA

The handbook has now been in the hands of readers for several months and has during that time been thoroughly checked over by many engineers. The errors to which attention has been called are listed below:

page 66, Table 3, Σo-values for $\frac{1}{4}\phi$ are in error. They should be equal to one-half of those given for $\frac{1}{2}\phi$.

page 89, first footnote should be changed to: Design spiral column as a round column but build it as a square column with the same nominal concrete dimension.

page 102, footnote: N should be $1000N$.

page 109, Table 38, Spiral Rods, $\frac{5}{8}\phi$: 0.25 should be 0.31.

page 113, Table 41, first column, third case: Denominator in equation for y should be $6[2bt + c(a + b')]$

page 120, Final coefficients: $+.069$ should be $-.069$.

Discussion to close in* Supplement *to be issued with September 1940* JOURNAL, *should reach A. C. I. Secretary by June 1.

Vol. 36 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN *APRIL 1940*

Job Problems and Practice

Some will ask questions — Some will answer them — Some will do both

A. C. I. members are invited to use this new JPP department as an informal means toward mutual help. When a problem holds possibilities of general interest the discussion will be briefed in these pages.

If you have a problem—present it; a question, ask it. If you know the answer to a question asked, or if you can contribute something which may help in the solution, or have reason to disagree with the answers or suggestions published, your contribution will be welcome.

If you know of an interesting problem whose solution has already smoothed the way of someone in the field, tell us about it—some other A. C. I. member may need just that information.

The "answers" are the answers of individuals to whom the questions are referred and not of the A. C. I. as an organization.—EDITOR

Design of reinforced concrete columns under eccentric Loads (36-58)

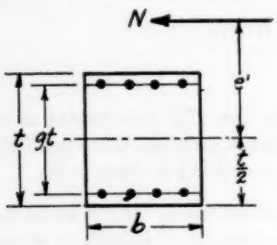
Q—Is there a simple and direct method for designing reinforced concrete columns subject to axial and binding stress?

A—The design of an eccentrically loaded column usually requires the assumption of a column section, an analysis to determine the stresses due to the given loading, and a re-design to meet the allowable stresses specified. The problem is greatly simplified by use of the Reinforced Concrete Design Handbook of A. C. I. Committee 317, recently issued by the Institute, to which references are made herein. The following examples illustrate the procedure to be followed in using the forthcoming 1940 Joint Committee Report.

The allowable fiber stress f_p, for eccentrically loaded columns under these authorities varies with the eccentricity, between f_a, the allowable concrete stress in an axially loaded column (with zero eccentricity) and f_b, the allowable concrete flexural stress (with infinite eccentricity).

Reinforcing steel is especially effective in resisting flexure, particularly when the effects of shrinkage and plastic flow are visualized. Hence it is logical in designing for eccentricity to use more than the minimum specified steel percentage of 1 per cent; from 2 to 4 per cent is preferable from the viewpoint of structural efficiency.

1—*Rectangular Tied Column*



$$p = \frac{A_s}{bt} = \frac{A_s}{A_g}$$

FIG. 1

Given: f'_c = 3000 p.s.i. n = 10
f_s = 12,800 p.s.i. for int. grade steel
N = 200 kips.
M = 100 ft.-kips.
$e' = M/N$ = ½ ft. = 6 in.
Assume b = 22 in. t = 22 in.
p = 0.02
(Note: This is facilitated by reference to Table A, which applies to uncracked sections. Taking $6\frac{e'}{t}$ as 2, $\frac{N}{A_g}$ is found to be 398. A_g = 502 sq. in, $b = t$ = 22.4 in.)

With t = 22 in., $\frac{e'}{t} = \frac{6}{22} = .27.$ $g = \frac{18}{22} = .82$

From Table A, $f_a = \frac{.18f'_c + 12{,}800p}{1 + (n-1)\,p} = \frac{540 + 256}{1.18} = 675$ p.s.i.

$$\frac{f_a}{f'} = 0\ 225$$

From A. C. I. Handbook, Table 33, D = 5.2, $\frac{De'}{t} = 5.2 \times .27 = 1.40$

From Handbook, Table 34, f_p = allowable concrete stress = 950 p.s.i.

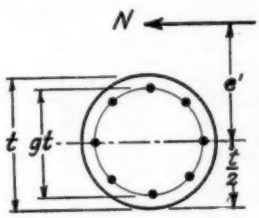
Compute $Q = \frac{A_g f_p}{1000\ N} = \frac{484 \times 975}{200{,}000} = 2.30.$ From Diagram 30, Handbook, with $\frac{e'}{t} = .27$, Q = 2.30 and g = 0.82, the required value

of $np = 0.13$, or $p = .013$. This could be used, but a larger percentage is preferable.

Assume column 20 in. square ($b = t = 20$ in.) and $p = .025$. Then $g = \frac{16}{20} = .8, \frac{e'}{t} = 0.30, D\frac{e'}{t} = 5.15 \times .3 = 1.54$. $\frac{f_a}{f'_c}$ is still 0.225

From Table 34, $f_p = 975$. $Q = \frac{400 \times 975}{200{,}000} = 1.95$. From Diagram 30, with $\frac{e'}{t} = .30$, $Q = 1.95$ and $g = 0.80$, the required value of $np = 0.30$ and $p = .03$. $A_s = 12.0$ sq. in.; use 12 1-in. sq., 6 in. each column face. (Check, 5 spaces at 3 in., 2 at $(1\frac{1}{2} + \frac{1}{2}) = 19$ in. O.K. in 20-in. column.) Ties, $\frac{1}{4}$ in. rd. at 12 in. This design appears preferable to the first one.

2—*Circular Spirally Reinforced Column.*



$p = \frac{A_s}{A_g} = \frac{4A_s}{\pi t^2}$

FIG. 2

Given: $f'_c = 3000$ p.s.i. $n = 10$
$f_s = 16{,}000$ p.s.i., int. grade steel
$f'_s = 40{,}000$ p.s.i., int. grade spirals
$N = 200$ kips.,
$M = 100$ ft.-kips.
$e' = 6$ in.
Assume $t = 22$ in. and $p = .03$
(From Table A, with $8\frac{e'}{t} = 2$, $A_g = 405$ sq. in. $t = 22.7$)

With $t = 22$ in., $\frac{e'}{t} = .27$, $g = \frac{18}{22} = .82$. From Table A, $f_a = \frac{225f'_c + 16000}{1 + (n-1)p} = \frac{675 + 480}{1.27} = 910$ p.s.i. $\frac{f_a}{f'_c} = 0.303$ $D = 7.4$ $D\frac{e'}{t} = 2.00$. From Table 34, Handbook, $f_p = 1150$ p.s.i.

Compute $Q = \frac{A_g f_p}{1000\ N} = \frac{380 \times 1150}{200{,}000} = 2.19$. From Diagram 32, Handbook, with $\frac{e'}{t} = .27$, $g = .82$ and $Q = 2.19$, the required $np = 0.42$ and $p = .042$. $A_s = .042 \times 380 = 16.0$ sq. in. Use 16 1-in. sq.

TABLE A

ALLOWABLE AVERAGE UNIT LOADS, $\frac{N}{Ag}$, FOR ECCENTRICALLY LOADED COLUMNS

Based on uncracked section, for use in preliminary design.

$$\frac{N}{Ag} = \text{Allowable unit load for eccentric loading} = \left[\frac{\text{Allowable unit load for axial loading}}{1 + \left(\frac{f_a}{.45f'_c}\right) D\frac{e'}{t}}\right]$$

$\frac{f_a}{.45f'_c} = \frac{\text{allowable concrete stress, axial column}}{\text{allowable concrete stress, in flexure}}$

$f_a = \frac{.225f'_c + f_s p}{1 + (n-1)p}$, for spiral columns, $= \frac{18f'_c + .8f_s p}{1 + (n-1)p}$, for tied columns

$D = 6$ for rectangular columns, 8 for circular columns, for preliminary design.
Allowable axial loads are according to 1940 Joint Committee Report.
This table applies for intermediate grade steel, for which $f_s = 16{,}000$ p.s.i.

f'_c	n	Values of $\frac{f_a}{f'_c}$ — Spiral Columns						$\frac{f_a}{f'_c}$ —Tied Columns			
		p = .01	.02	.03	.04	.05	.06	.01	.02	.03	.04
2000	15	.267	.300	.328	.350	.383	.407	.214	.240	.262	.280
2500	12	.261	.289	.314	.334	.367	.392	.209	.232	.252	.268
3000	10	.255	.281	.304	.322	.354	.380	.204	.225	.242	.258
4000	7.5	.249	.270	.289	.305	.335	.359	.199	.216	.231	.244
5000	6	.243	.262	.279	.294	.321	.344	.196	.210	.223	.235
f'_c	$D\frac{e'}{t}$	Values of N/Ag — p.s.i. Spiral Columns						Values of N/Ag — p.s.i. Tied Columns			
		p = .01	.02	.03	.04	.06	.08	.01	.02	.03	.04
2000	0	610	770	930	1090	1410	1730	488	616	744	872
	0.5	471	578	682	785	989	1191	394	486	576	665
	1	383	462	538	613	762	909	331	402	470	538
	2	279	330	379	427	522	616	250	298	344	389
	4	181	210	238	265	320	375	168	197	224	250
2500	0	723	883	1043	1203	1523	1843	578	706	834	962
	0.5	560	668	773	877	1082	1284	469	561	652	741
	1	458	537	614	690	839	986	395	466	535	603
	2	335	386	435	484	579	673	299	348	394	439
	4	218	247	275	303	358	411	202	231	258	285
3000	0	835	995	1155	1315	1635	1955	668	796	924	1052
	0.5	651	758	864	968	1174	1375	544	637	728	817
	1	533	613	690	767	915	1060	459	531	601	669
	2	392	443	492	541	636	727	350	398	445	490
	4	256	285	313	341	395	447	237	265	293	320
4000	0	1060	1220	1380	1540	1860	2180	848	976	1104	1232
	0.5	831	939	1045	1150	1356	1558	695	787	878	969
	1	683	763	840	918	1067	1213	588	659	730	799
	2	503	554	604	654	748	840	450	498	545	591
	4	330	359	387	415	468	421	306	334	362	389
5000	0	1285	1445	1605	1765	2085	2405	1028	1156	1284	1412
	0.5	1012	1119	1224	1330	1536	1739	844	937	1029	1120
	1	834	913	990	1068	1217	1363	716	788	859	928
	2	617	667	716	765	859	951	549	598	645	691
	4	406	434	461	489	541	592	375	403	431	457

bars. Spiral required, $p' = 0.45(R-1)\frac{f'_c}{f'_s} = 0.45\left(\frac{380 - 284}{284}\right)\frac{3}{40}$ $= .0114$. Use $\frac{3}{8}$ at 2 in. spacing (Table 26, or 37, Handbook).

Somewhat less vertical reinforcement could be used with a 23 in. or 24 in. column. Assume $t = 23$ in. and $p = .03$. $\frac{e'}{t} = .26$. $g = \frac{19}{23}$

$= .825$. $D = 7.42$ $D\frac{e'}{t} = 1.93$. $\frac{f_a}{f'_c} = .303$, $f_p = 1160$, $Q = \frac{415 \times 1160}{200{,}000} = 2.41$. From Diagram 32, $np = 0.29$ and $p = .029$. $A_s = .029 \times 415 = 12.1$ sq. in. Use 16 1-in. rd. bars. Spirals $p' = 0.45\left(\frac{415 - 314}{314}\right)\frac{3}{40} = 0.108$. From Table 37, use ⅜ rd. at 2 in.

From these examples it appears that Table A gives a very close approximation to the final column sizes chosen. Obviously, it will be less accurate as the eccentricity increases, and since it is intended only for preliminary use, large values of $D\frac{e'}{t}$ are not included. However, for eccentricities less than half of the column depth, Table A could be used for final design without great inaccuracy.

Only in cases involving a very large eccentricity is it necessary to investigate the tensile stress in the column steel. The tensile stress in a column may easily be determined by use of Diagrams 30 to 32 of the Handbook, using the relation $f_s = nf_p\left(\frac{1 + g}{2k} - 1\right)$. Since the value of nf_p will never exceed 13,500 p.s.i., and generally will not exceed 10,000 p.s.i., it is found that with an allowable value of $f_s = 20{,}000$ p.s.i., the matter of tensile stress need be checked only when the value of kt (distance from neutral axis to compression surface) is less than about $0.35\ t$. In the examples given above k varied from 0.7 to 0.8 and the tensile stresses in the steel were far below the allowable values.—F. E. RICHART.*

Compressed Air Jets as a means of Placing Concrete (36-54)†

In 1925-26 during the construction of pier No. 6 at the Navy Yard, Puget Sound, Bremerton, Washington, we used an air jet to settle the concrete into the forms and around the reinforcing steel for all the deck concrete, over 15,000 cu. yds. The mix was 1:2:4¼ and the slump was 3 to 4 in.

The jet was very simple, a short length of ¼ in. pipe with a standard cap on the jet end. Just above the cap were two 40 gauge holes diametrically opposite each other. A valve regulated the flow of air.

*Research Professor of Enginering Materials, Univ. of Illinois.

†See previous contributions to this subject, J.P.P. section, A. C. I. JOURNAL, Jan. 1940, p. 317; Feb., p. 413.

The use of the jet was not original on this work, but was brought to the job by a concrete worker who had seen it used in Alaska. There was some skepticism about its use, and several simple experiments were made to demonstrate that the concrete was not left holding a lot of air bubbles. This was before the days of vibrators,, as we now have them, and it is believed this use was well worth while. Naturally, judgment was required in their use, or more harm from segregation would result, than good was accomplished—the same however, is true of our present day vibrators. We found that the jet was especially useful in getting the concrete to flow in and around the bottom beam bars and into sharp corners of the form. Personally I believe that the results which were obtained, were better than they would have been by tamping alone.

In our work we did not even come near the method described by Mr. Ruettgers in his comment on this subject, in which he left the air jet on for five minutes. We kept the jet on the move all the time with the valve just "cracked"; in fact the procedure was very similar to the use of the vibrators of today.

Personally I consider the vibrators superior to air jets; but at the same time, jets can be used successfully.—W. F. WAY†

†Vice-pres., Stuart Cameron & Co., general contractors, Vancouver, Canada.

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *APRIL 1940*

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Soil-cement revetment moves

Engineering News-Record, Vol. 123, No. 15, Oct. 12, 1939, p. 53. Reviewed by S. J. CHAMBERLIN

Despite the slippage of underlying stratum due to softening by seasonal floods, a river-bank revetment of cement-stabilized soil in sacks has held together well through movements of 5 ft.

A cotton warehouse in Dugaresa (Yugoslavia)

FRITZ EMPERGER, *Beton und Eisen*, Vol. 38, No. 17, p. 275, Sep. 5, 1939. Reviewed by A. U. THEUER

The author describes a three-unit warehouse in which centrifugally-spun concrete-pipe beams were used. This construction was chosen to illustrate the advantages to be gained in the use of centrifugally-spun concrete units.

Reinforced concrete hinges and stilts

DR. GAEDE, *Beton und Eisen*, Vol. 38, No. 11, p. 185, June 5, 1939. Reviewed by A. U. THEUER

Tests were made at the Hanover Technical University to check a method of stress analysis proposed by Dr. Bortsch for the design of reinforced concrete hinges. A steel plate placed on edge was loaded over a very short length of one edge and stress trajectories over the flat side were determined. Good agreement with the proposed method of stress analysis of Bortsch is claimed.

Portfolio of 1939 buildings

Engineering News-Record, Vol. 123, No. 17, Oct. 26, 1939, pp. 63-94. Reviewed by S. J. CHAMBERLIN

Included in a description of some of the notable buildings of the year are: the Armstrong tire factory (Anton Tedesko) of monolithic concrete construction with thin-shell multiple-barrel roof; the Castle Village apartments with new ideas in constructing tall concrete apartment buildings; and the Maxwell House coffee plant of heavy concrete flat slab construction.

Ultramicroscopic investigation of the hydration of lime

O. E. RADCZEWSKI, H. O. MULLER and W. EITEL, *Zement*, Vol. 28, No. 49, Dec. 7, 1939, pp. 693-697
Reviewed by L. T. BROWNMILLER

This article contains some interesting photographs of hydrated lime taken under the electron microscope, magnified up to 30,000 diameters. The calcium hydroxide

was obtained by separation from a saturated lime water and consisted of compact spherulites of diameters between 0.1-1 micron. Similar spherulites are formed on shaking cement with water.

Construction progress at Shasta Dam

Engineering News-Record, Vol. 123, No. 19, Nov. 9, 1939, pp. 48-51. Reviewed by S. J. CHAMBERLIN

With the excavation 80 percent complete and work progressing satisfactorily on the 460-ft. head tower for the seven radial cableways, concrete pouring should begin as planned on the completion of the railroad relocation. The completed plant is to deliver 6,000 cu. yd. of concrete per day, or 225,000 cu. yd. per month. Equipment is being assembled at the gravel pits to furnish four sizes of aggregate, 6-3 in., 3-1½ in., 1½-¾ in. and ¾-¼ in. Two grades of sand, 4-24 mesh and 28-48 mesh, together with rod-mill ground fines 48-100 mesh will constitute the fine aggregate.

The development of masonry and reinforced concrete bridges

A. KAISER, *Beton und Eisen*, Vol. 38, No. 14, p. 225. Reviewed by A. U. THEUER

The author, after a brief review of the notable masonry arches and massive bridges of the Romans and their renascence in more recent times, engages in a detailed description of the modern reinforced concrete bridge in Germany. By means of selected examples giving general dimensions, he describes the unique features of a variety of beam, cantilever, frame, and arch structures. With 49 photographs, cuts and drawings the article is well illustrated.

A stress-cured concrete pavement

F. W. RUSSE, JR., *Engineering News-Record*, Vol. 123, No. 19, Nov. 9, 1939, pp. 68-69
Reviewed by S. J. CHAMBERLIN

Two 940-ft. experimental lengths of concrete pavement were cured with the slabs under endwise compression to determine if tension cracks due to shrinkage and temperature changes could be prevented. Transverse joint forms (spaced from 40 to 100 ft. apart) were removed shortly after initial set and collapsed hose forming rubber pressure cells were placed in each ¾-in. space. Pressure was then applied to the cells in increasing intensities for 5 days. Safe pressures were determined by applying destructive pressures to adjacent small test slabs. Service behavior as compared with nearby unstressed pavement is yet to be reported.

Durability investigations

A. T. GOLDBECK, (National Crushed Stone Association), Highway Research Board paper, Dec., 1939.
HIGHWAY RESEARCH ABSTRACT

The purpose of this paper is to show the need for a rather comprehensive investigation of the phenomenon of lack of durability of concrete. At the present time, freezing and thawing tests are made in different laboratories, but the methods differ widely and they give different results. Similarly, aggregates are tested with various forms of artificial accelerated soundness tests. It is believed that certain factors regarding aggregates and regarding conditions of exposure are being overlooked in connection with laboratory investigations of soundness. Attention will be called to some of these phenomena with the purpose of stimulating discussion which may lead to a clarification of the needs in connection with soundness testing.

Ross Dam built in isolated Skagit Gorge

Engineering News-Record, Vol. 123, No. 15, Oct. 12, 1939, pp. 48-52. Reviewed by S. J. CHAMBERLIN

The first stage of Ross Dam is being built in an isolated and hard-to-make-accessible site. Men and material, with the exception of aggregate, are transported over

private railroad, incline, and by boat. The present work is the lower half of a high, variable-radius, concrete arch dam to be completed later. Provision is made for a third development for converting the dam into a curved gravity-section 653 ft. high. Space for the construction plant was carved out of side of the canyon or built up of timber cribbing and trestles. The plant layout includes a high-line cableway with traveling head tower, full automatic mixing plant, belt conveyor system for aggregate and suspension bridges for trucks and conveyor.

Point-supported dome of thin shell type

ANTON TEDESKO, *Engineering News-Record*, Vol. 123, No. 23, Dec. 7, 1939, pp. 85-86.
Reviewed by S. J. CHAMBERLIN

A 110-ft.-diameter dome forms the roof of the central hall of a new auditorium at Tulane University. Restricted by the plan to a minimum column spacing of 54 ft., the designer chose a thin curved slab stiffened by polygonal diaphragms. Tests with a rubber model indicated that the diaphragm members in combination with the slab took full load as long as the portion of the ring girders outside the diaphragms were flexible enough to move horizontally. The design, therefore, provided for a flexible ring girder outside of the diaphragm walls and a stiff ring girder between intersections of the individual walls with the girder.

Water-power dam in Sweden formed of concrete monoliths

Engineering News-Record, Vol. 123, No. 21, Nov. 23, 1939, pp. 63-64 Reviewed by S. J. CHAMBERLIN

This article is a condensation of the original paper in *The Engineer* (London). The main dam consists of T-shaped concrete monoliths poured in place, the heads being in contact form a solid upstream face. The space between the stems results in decreased concrete yardage offsetted by increased form costs. The spillway section is similarly formed from I-shaped monoliths. Advantages claimed are: (1) simple construction of gravity type; (2) reduction of uplift; (3) simplification of the conditions for stability; (4) preventation of percolation through concrete; (5) accessibility for inspection and repair; (6) reduction of shrinkage and temperature cracks; and (7) easy control of the generation and dissipation of the chemical heat in the concrete.

Certain details of construction in long-span reinforced concrete bridges

DJORDJE LAZAREVIC, *Beton und Eisen*, Vol. 38, No. 17, p. 277, Sept. 5, 1939.
Reviewed by A. U. THEUER

The author refers to the publications of Hawrenek regarding the reduction of load on the falsework during the period of construction of long span concrete arches. He then advances another method of construction whereby the arch breadth is divided up into n-pairs of individual arches. Each pair of arches is poured separately and upon hardening lifted off of the falsework by means of hydraulic presses before the following pair is poured. After completion of all the pairs, they are joined together and the roadway poured. An advantage for this method is claimed on the basis of the greater certainty regarding the stress distribution between the different elements of the cross-section. The unique element in the author's method consists in building the hydraulic presses into the falsework and not the concrete arch. The possibility of buckling of the thin sections is thereby avoided.

The author then discusses various types of long span beam constructions in regard to the prestressing of the steel in place and the varied uses to which standard hydraulic jacks may be put.

Elliptical concrete domes for sewage filters

ERIC C. MOLKE, *Engineering News-Record*, Vo. 123, No. 19, Nov. 9, 1939, pp. 59-61.
Reviewed by S. J. CHAMBERLIN

The two thin-shell domes, 150 ft. in diameter and 32 ft. high, were designed with an elliptical cross-section, giving vertical shell walls at the base to eliminate horizontal thrusts. With no base ring at the top of the heavy sub-structure the shell has sufficient flexibility for temperature differences. A heavy ventilation lantern at the top and louvres around the base complicated the dome design. 1-in. boards warped into place over curved wooden ribs supported by timber rings over posts formed the interior form. Outside forms were needed only on the lower 12 ft. 4-in. slump concrete with an upper layer of 2-in. slump concrete gave satisfactory bottom surfaces, the top being screeded and belted. On completion the outside of the shell was given two coats of emulsified asphalt followed by one coat of aluminum paint.

Application of vibrators for measuring mortar consistency and fabricating mortar cubes

RAYMOND L. BLAINE and JOHN TUCKER, JR., Research Paper RP1273—National Bureau of Standards, Vol. 24, Feb., 1940 AUTHORS' ABSTRACT

Two vibrators—one magnetic, with linear vertical motion; one mechanical with rotary horizontal motion—were employed to compact mortars of varying compositions, for various durations and at various frequencies and amplitudes. The effectiveness of the treatment was judged by (1) the reproducibility of the average strength of the six cubes in one batch, (2) the reproducibility of the strengths of individual specimens, and (3) the appearance of the compacted specimen. Standard Ottawa sand mortars possessed advantages over those with two or more types of aggregate. The mechanical rotary vibrator, designed for the investigation, was found to give better results than the magnetic vibrator. Either vibrator may be used to measure the consistencies of mortars of the dryness suitable for compaction by vibration.

Bond tests on rusted bars

H. J. GILKEY, S. J. CHAMBERLAIN, and R. W. BEAL, (Iowa State College), Highway Research Board paper, Dec., 1939 HIGHWAY RESEARCH ABSTRACT

Plain round $\frac{5}{8}$ in. steel bars were exposed to the weather for 0, 1, 2, 3, 4, 6, 7, and 8 months respectively. Rust formation was observed and measured by abrading the rust and weighing it. On companion bars the rust was: (a) left unmolested, and (b) wiped off with burlap. All bars were vertically cast into 10-in. 28-day pullout specimens ($L/D = 16$) and observations were taken for "first slip," ultimate bond resistance and drag resistance after a slippage of 0.08 in. Amounts of slippage were measured at both the loaded and the unloaded ends of the bars. The general conclusions reached are that the light layer of loose powdery red rust that first forms is of negligible importance and that firm rust is not detrimental to bond resistance and may actually increase it. Wiping rusted bars off with burlap had no important effect, the tendency in these tests being to decrease the bond somewhat for early or short rusting periods and to increase the bond resistance appreciably where the rust had become deep, loose and flaky as it was for the bars which were exposed for eight months. In the eight-month exposure the rust was so deep that it was making inroads on the effective cross-section of the bar and was beyond what would be considered permissible regardless of its effect on the bond resistance.

Durability Investigation

A. F. GILL, (National Research Council of Canada), Highway Research Board paper, Dec., 1939.
HIGHWAY RESEARCH ABSTRACT

The author advances the interesting hypothesis that the nature of frost heave in roads, the spalling of concrete pavement, and the disruption of masonry through frost action is substantially the same. It has been established that frost boils and the greater part of frost heaving is a result not so much of freezing of moisture present in any one spot in soil capillaries but of the building up of ice plates through migration of moisture from road shoulders and sub-grade water tables.

It is the author's belief that the spalling of concrete and masonry is due not to the freezing of static water within the pores of the mass, but to the building up of ice plates or incipient ice plates due to migration from the warm side. To investigate such a conclusion, a test was devised that duplicated as nearly as possible the natural conditions. As a first step, standard building bricks were tested with one end in water above the freezing point, the other end exposed to a temperature between 0 and 10 deg. F., and the mid-section of the brick surrounded by insulation.

Preliminary tests on a porous brick seem to bear out the assumptions, for the brick spalled quickly near the cold end, producing a clean fracture which was coated with about 1 mm. of continuous ice.

Experiments in curing concrete pavements

BERT MYERS, (Iowa Highway Commission), Highway Research Board paper, Dec., 1939.
HIGHWAY RESEARCH ABSTRACT

Thirteen years of experience in curing concrete pavements with a great many types and kinds of cures form the background for this report. Among those methods used were wet earth, wet burlap, various bituminous types, calcium chloride both externally and interally, the Hunt process and others. From these several experiments a few brief conclusions were drawn as follows:

1. Earth and water curing provides the most desirable results of any that were studied.
2. The curing treatment applied during the first few hours after the concrete is placed has the greatest effect upon its strength.
3. The effect of a curing method upon strength may be predicted from laboratory tests for prevention of moisture loss.
4. While under the climatic conditions existing in Iowa, differences of strength due to differences in curing methods tend to decrease with the age of the concrete, they do not disappear entirely for a very long time.
5. The difference in slab length due to differences in methods of curing tend to disappear with increasing age of the pavement.

Stresses in concrete runways of airports

H. M. WESTERGAARD, (Harvard University), Highway Research Board paper, Dec., 1939.
HIGHWAY RESEARCH ABSTRACT

The problem of stresses in concrete runways of airports is essentially the same as the problem of stresses in concrete pavements of highways, except that both the wheel loads that must be contemplated and the areas of contact between tire and concrete are greater for runways than for highways. These differences, especially the greater areas of contact, call for a re-examination of the formulas that have been derived for stresses in concrete pavements due to wheel loads. The study is limited to the case of loads at an appreciable distance from edges or joints.

The report of results is a report of "no surprises." The larger area of contact between the concrete pavement of the runway and the tire of each wheel of the

landing gear, as compared with the corresponding areas for tires of trucks on highways, call for only very small revisions of the formulas presented by the writer previously, in the *Proceedings* of the Highway Research Board, 1926, and in *Public Roads*, April 1926 and December 1933. In fact, the needed revisions, of the formulas are so small compared with the well-realized uncertainties, including those of the sub-grade, that in most cases the corrections may be ignored. Yet the formulas for the corrections are simple and not difficult to use. In the interest of recording the evidence the fairly simple derivations of the formulas are shown at the end of the paper.

Underpinning a postal building in Konstanz

H. DORR, *Beton und Eisen*, Vol. 38, No. 16, p. 257, Aug. 20, 1939. Reviewed by A. U. THEUER

A description of a post office building erected in 1891 and subject since early in construction to serious settlement is here presented. The foundation consisted of a concrete mat (built before the days of reinforced concrete) with embedded I-beams under the heavy masonry-faced exterior walls. The sub-strata of the entire surrounding area is an old lake bottom composed of sedimentary deposits of clay. So serious was the settlement during construction—27.8 cm in a little over one month's time—that the coursed sandstone facing had to be altered in design. Although with a constantly decreasing increment, settlement is still progressing. Due to the unequal loads superimposed by the faced street walls and unfaced court-yard walls the settlements were very unequal. After serious consideration of wrecking the building, a plan of underpinning proposed by the author of this article was decided upon as an experiment. Short sections of reinforced concrete piles were forced down by hydraulic-jacking to a point where the resistance to further driving seemed satisfactory. One hundred and thirty-eight piles were used. The details of securing separate sections of each pile, one on top of another, to obtain transverse rigidity is shown by means of drawings. Some detailed records were also kept regarding the rates of driving and resistances, the spring-back or elasticity of the foundation material and variations resulting from different methods of driving.

The effect of sulphate on the shrinkage of portland cement

G. HAEGERMANN, *Zement*, Vol. 28, Nos. 40, 41, Oct. 5, 12, 1939, pp. 599-602, 609-614.
Reviewed by L. T. BROWNMILLER

The data reported here are on the effect of the following sulphates on the shrinkage of portland cements: Na_2SO_4, K_2SO_4, $FeSO_4$, $MgSO_4$ and $CaSO_4$. In addition to the shrinkage measurements, strengths, setting time, and water loss under various conditions of storage were followed. The sulphates were added to cements containing the normal amount of gypsum.

The alkali sulphates reduce shrinkage up to 28 days but at later ages the shrinkage is greater than with corresponding amounts of gypsum. Ferrous sulphate causes excessive shrinkage. Magnesium sulphate reduces the shrinkage of cements with high tricalcium aluminate and increases the shrinkage of cements with low tricalcium aluminate. Calcium sulphate acts similarly to magnesium sulphate in its effect on the shrinkage of cements. Considering the total effect of the additional sulphate on all of the properties of the cement, calcium sulphate gives the most favorable results. However the best amount to be added must be determined for each cement.

The author makes one very interesting deduction from the results of his work. Upon making these additions of sulphate to normal cements the total SO_3 is in-

creased beyond the limit permitted in the specifications, but in view of the fact that favorable changes in the properties of the cement are obtained he states that a change in the specifications might be desirable.

Current research bearing on structural design of concrete pavement

A. A. Anderson (Portland Cement Association), Highway Research Board paper, Dec., 1939.
Highway Research Abstract

This survey of current research in the State highway departments includes all work dealing in any way with concrete. On the 84 projects being conducted by 25 departments at the present time, none was devoted to measurement of stresses in pavements or to the performance of varying cross-section under traffic. The majority of the reported investigations deal with the subject of joints. Among problems being investigated are load transfer devices, dummy joints, joint fillers, spacing of joints, and the expansion space requirements. Another group of investigations which deals with the improvement of concrete mixes, includes determination of the effects of types of cement and admixtures, investigation of low cement factors, etc. Numerous investigations are also being made of the characteristics of concrete, such as determination of modulus of elasticity, effects of temperature, vibration, curing methods, and many others. Also included in this survey are a number of investigations of the subgrade as related to bearing power, which have effects on the design of concrete. A number of miscellaneous studies are also listed, such as condition surveys, placement methods, and others. The report also makes suggestions for future research work which are in general along the lines of current investigations, but of a more detailed nature or covering a wider range of conditions.

Building the world's longest floating bridge

Charles E. Andrew, *Civil Engineering*, Vol. 10, No. 1, January, 1940, p. 17.
Reviewed by J. R. Shank

The concrete pontoons forming sections of the Lake Washington pontoon bridge, now nearly finished are from 117 to 378 feet long by 59 feet wide by 14 ft. 6 in. deep. The bottoms, sidewalls, and most of the top deck are 8 in. thick. Three 6 in. partition walls divide the pontoon longitudinally into 4 cells. Cross walls divide the length so as to form more or less cubical cells. These pontoons are cast in graving docks and then floated to the bridge site. The concrete used has a slump of from 5½ to 6 in. and a compressive strength of 3500 p.s.i. at 27 days. The bottoms, sidewalls and partitions are poured monolithically and the deck poured at a later date. Collapsible wood forms are used.

Anchors are of three types, two of which are concrete. The first one, of concrete, weighing 65 tons is of such a shape as to act similar to that of a steel ship's anchor and is sunk into soft bottom by means of high-pressure jets. The second is a dead weight anchor, a concrete box weighing 75 tons filled with sand and gravel after being sunk to its final position. It is further held in place by 800 cu. yd. of quarry rock placed in front of it. The third is a double stake and guy anchor. Both stakes or piles are of steel, 2-24 in. WF x 14 in. at 130 lb., sections fabricated together and are driven by means of a pile driver. The anchor guy, from one pile to the other, is 3½ in. diameter forged steel with a forged steel turnbuckle.

At the westerly end, four pontoons act as a unit to a hinge at the end of the fourth and are provided with bilge pumps for water ballast for the purpose of adjusting the level to suit a 3 ft. seasonal rise and fall of the lake. At the rigid connections of pontoon to pontoon, large shear keys of concrete, male and female, serve to main-

tain alignment. Large bolts (3½ in. dia. upset) around the top, bottom, and sides hold the ends together. They are pulled up, compressing a rubber gasket until the space between the pontoon and walls is one inch. This one inch space inside of the gasket is filled with cement grout and the bolts are later tightened.

Light-reflecting characteristics of pavement surfaces

G. S. PAXSON and J. D. EVERSON, Technical Bulletin No. 12, Oregon State Highway Commission, Dec., 1939. AUTHORS' ABSTRACT

Tests were made to determine the amount and distribution of the light reflected from various types of pavement surfaces with especial reference to the glare from automobile headlights. Tests were made on 9 actual road surfaces and on 20 small-sized panels. The surfaces included portland cement concrete ranging from steel troweled finish to ribbed or grooved finish, asphaltic concrete, bituminous macadam, and oiled macadam of fine, medium, and coarse textures. Beams of light were directed onto the pavement surfaces at varying angles, and the intensity of the reflected light in different directions was measured by a light meter. Both dry and wet surfaces were tested.

Very little glare was found on any dry surface. The light-colored portland cement concrete pavements were found to be about 4 times as luminous as the dark asphaltic types. The rough-textured macadam surfaces reflected more light back toward the source than away from it.

When the surfaces became wet, the glare increased by varying amounts depending on the type of surface. The glare was most intense from the smooth-textured pavements such as smooth or worn portland cement concrete and fine-textured asphaltic concrete. Some of the open-type asphaltic concretes and coarse-textured bituminous macadams gave no more glare when wet than when dry. The glare from deeply-broomed finish portland cement concrete was not objectionable.

The glare from a wet pavement surface comes from the film of water on the surface and is, therefore, dependent upon the ability of the surface to drain the water away before it is flooded. The flatter the pavement surface and the heavier the rainfall, the better the drainage must be to prevent flooding. The medium-coarse-textured surfaces, such as the deeply-broomed concrete, the open-type asphaltic concrete, and the bituminous macadam surfaces, were satisfactory under rainfall up to a rate of 1 inch per hour.

Experiments with continuous reinforcement in concrete pavements

EARL C. SUTHERLAND and SANFORD W. BENHAM, (Public Roads Administration and Indiana Highway Commission), Highway Research Board paper, Dec., 1939 HIGHWAY RESEARCH ABSTRACT

It has been suggested that longitudinal steel reinforcement might be used in concrete pavements as a means of reducing the number of transverse joints and objectionable transverse cracks.

A limited study of the use of steel reinforcement for this purpose has been in progress over a period of years by the Public Roads Administration. It has been observed that while numerous transverse cracks occur in such a pavement, the cracks remain closed and the pavement remains structurally sound.

In 1936 the Bureau of Public Roads and the Indiana State Highway Commission decided to make an extensive cooperative investigation of the use of reinforcement in concrete pavements using a wide range in the amounts of steel. For this purpose a series of test sections on approximately six miles of pavement was constructed in

1938 on a main highway in the State of Indiana. The maximum amount of longitudinal reinforcement used was 1-inch diameter bars spaced at 6-inch centers. (534 pounds of longitudinal steel per 100 square feet.) The minimum amount was No. 6 wire (diameter 0.192-inch) spaced at 6-inch centers. (22 pounds of longitudinal steel per 100 square feet.) The lengths of the slab units were adjusted depending upon the amount of longitudinal steel and varied between 20 and 1320 feet.

This pavement is approximately one year old and has been under heavy traffic during a large part of the time since it was built. A considerable number of very fine transverse cracks have occurred in the central portion of the long heavily reinforced sections. In the short sections reinforced with small amounts of steel, only a limited amount of cracking has developed.

Skid-resistant characteristics of Oregon pavement surfaces

JOHN BEAKEY, K. M. KLEIN, and W. J. BROWN, Technical Report No. 39-5, Oregon State Highway Commission, Dec., 1939. AUTHORS' ABSTRACT

Tests were made to determine the skid-resistant properties of representative pavement surfaces on the Oregon highway system. Tests were made on 36 surfaces grouped into four general classes: portland cement concrete, asphaltic concrete, bituminous macadam, and oil-mat pavements. The pavements ranged in age from worn pavements laid in 1920 to new pavements laid in 1938 and tested before being subjected to traffic. A wide variety of surface textures was included in the test series, ranging from overly-fat asphaltic concrete pavement, where the bleeding of the asphalt had covered the surface with free asphalt, and portland cement concrete pavements worn smooth by 17 years of travel to new deeply-broomed portland cement concretes and open-textured asphaltic concretes and bituminous macadams.

The tests were made with a specially-constructed trailer connected through a dynamometer to the towing vehicle. The tractive resistance was determined by locking one wheel of the trailer while maintaining a constant speed with the towing vehicle. The tractive resistance is reported in terms of the coefficient of friction. Tests were run at speeds of 10, 20, 30, and 40 miles per hour. The surfaces were tested both in the dry and wet condition. A wheel load of 800 pounds was used in the main test series with a few representative surfaces retested with 400-pound wheel loads to determine the effect of weight.

The coefficient of friction was found to exceed the generally-accepted design standard of 0.40 on all the surfaces tested when these surfaces were dry. At high speeds and with wet surfaces a wide variation in skid resistance was observed. The smooth-textured surfaces, when wet, had coefficients of friction of under 0.30 and in a few instances of under 0.20. The deeply-broomed portland cement concretes, open-type asphaltic concretes, and bituminous macadams were only slightly affected by water. Those surfaces which permitted the water to drain away before the surface flooded gave the most skid resistance when wet.

Speed and wheel load were found to be important factors affecting skid resistance. An increase in speed from 10 to 40 miles per hour decreases the coefficient of friction from 20 to 30 per cent. Doubling the wheel load resulted in a decrease in the coefficient of friction of from 20 to 40 per cent.

Testing the efficiency of concrete curing methods

HENRY D. DEWELL, *Western Construction News*, Dec. 1939. AUTHOR'S SUMMARY AND EXCERPTS

In connection with the third three million gallon reinforced concrete overhead water storage tank built for the City of Sacramento, Calif., tests were made to determine the efficiencies, as measured by strength, of three methods of curing

concrete: (1) spraying with a curing liquid; (2) covering with commercial curing paper, and (3) leaving the forms in place.

Comparisons were made of the strengths of test cylinders cured by the three methods with the strength of standard test cylinders, both laboratory-cured and job-cured, and with the strength of uncured (air-cured) cylinders. While few in number, the tests represent job, rather than laboratory conditions, and the test results are consistent. The paper tabulates temperature and humidity conditions during the tests (June to Oct.). Concrete was a 1—5.5 field mix with ⅞ in. max. size gravel aggregate (1 cement, 2.7 fine aggregate, 2.8 coarse aggregate with 6.19 sacks concrete per cu. yd. of concrete in place).

TABLE 3—RELATIVE EFFICIENCIES OF CURING METHODS

Method of Curing Cylinders	*Efficiency* *Col. 1*	*Col. 2*
Laboratory cured	100%	117%
Job cured by burying in damp sand	85	100
Job cured with colorless curing liquid	26	31
Job cured in wooden forms	82	96
Job cured in Sisalkraft paper	51	60

TABLE 4—RELATIVE STRENGTHS FROM CURING METHODS

Method of Curing Cylinders	*Efficiency* *Col. 1*	*Col. 2*
Laboratory cured	100%	107%
Job cured by burying in damp sand	94	100
Job cured with colorless curing liquid	71	76
Job cured in wooden forms	93	100
Job cured in Sisalkraft paper	81	86

The average strength of the uncured, or air-cured concrete cylinders was 2,800 p.s.i., and the average 28-day strength was 4,533 p.s.i., showing a gain in strength by curing of 1,733 p.s.i. Calling this increase in strength due to curing 100 per cent, and basing the efficiencies of the other methods on this figure of 100 per cent, the relative efficiencies are as given in Column 1 of Table 3.

If, on the other hand, the increase in strength due to curing, shown by the standard job-cured cylinders (1,463 lb.), is taken as 100 per cent, the relative efficiencies of the other curing methods become as shown in Column 2 of Table 3.

These curing efficiencies should not be confused with the relative strengths of the cylinders cured by the different methods. The relative strengths, as measured by the compressive strengths are summarized in Table 4, in which Column 1 gives the relative strengths based upon the standard laboratory-cured cylinders at 100 per cent and Column 2 gives the corresponding values based upon the job-cured standard cylinders.

The tests indicate the very low efficiency of the curing liquid, and the effectiveness of wooden forms, although unmoistened except for initial wetting, in preventing loss of moisture when tight contact against the concrete is maintained. It is to be noted that both the air-cured cylinders and those sprayed with the colorless curing liquid showed no increase in strength after 28 days. Also to be noted is the high ratio of the 7-day strength of the liquid-sprayed cylinders to their 28-day strength.

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Attempts to Measure the Cracking Tendency of Concrete*

By Roy W. Carlson†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

A method is described for determining the resistance to cracking of concrete subjected to drying. A column of concrete is restrained by a centrally-located bar of steel, which is threaded near its ends to provide good bond there while it is coated over its mid-length portion to prevent bond. When the concrete dries and shrinks enough to become stressed beyond its tensile strength under the test conditions, an easily visible crack occurs. A few preliminary test results are offered, which in some cases at least, seem to be more nearly in line with field observations than are the results of free-shrinkage tests.

INTRODUCTION

A large percentage of the cracks in concrete are recognized as being due to drying shrinkage. The slow loss of moisture due to evaporation from the surface is probably the main cause of cracking in most thin-walled structures, especially in dry climates.

Research workers have recognized the importance of drying shrinkage for many years. Thousands of specimens have been tested to measure the drying shrinkage of all manner of concretes. The results have added much to the knowledge of concrete but they have not solved the cracking problem. Not always does a concrete which exhibits a low measured shrinkage crack correspondingly little in a structure.

The ability of a concrete to withstand cracking depends upon several factors besides the extent of its tendency to shrink. First,

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there is the degree of restraint. If the concrete is free to shrink as it desires, it will not crack. Only when the shrinkage tendency is resisted does cracking occur. For the purpose of the present paper, however, the degree of restraint will be assumed to be a fixed quantity because the intention here is to compare different concretes under similar conditions.

A second factor affecting the ability of a concrete to withstand cracking is obviously the tensile strength. Other things being equal, the amount of potential shrinage which can be withstood is proportional to tensile strength.

Another factor is the "extensibility." This is a term which signifies the amount the concrete can be stretched. Due to the fact that concrete is not always stretched to the point of cracking, it is more convenient in computations to use the term "sustained modulus of elasticity," which signifies the sustained force required to produce a unit deformation. The latter term is exactly like ordinary modulus of elasticity, except that it includes the slow, plastic movement as well as elastic movement. Concrete which has the lowest sustained modulus of elasticity stretches the most due to a given force, and such concrete is likely to resist cracking better than a less extensible, brittle concrete.

Finally, there is the time factor which also affects cracking of concrete. The simplest example of the effect of time is the well known fact that concrete can be deformed but little without cracking if the deformation is suddenly applied. Thus a concrete which shrinks a given amount over a long period of time would be expected to withstand cracking much better than one which shrinks the same amount more quickly.

THE TEST FOR CRACKING

Realizing that the tendency of a concrete to crack was dependent on many factors, a test was devised to measure the tendency to crack more-or-less directly. The essential feature of the test was to note the time required for restraned specimens of concrete to crack. A column of concrete was cast around a bar of steel in such a way that as the concrete tended to contract it was resisted by the steel. The concrete was bonded to the steel bar near the ends only, with the help of screw threads on the steel. For a length of a foot or more in the central portion, the steel bar was wrapped with cloth to prevent bond in this region. When a crack occurred in the concrete, it relieved the stress for the entire length between the threaded ends of

the steel bar. The crack was therefore of appreciable size and could be detected readily.

The procedure was to keep the concrete moist for a fixed length of time after casting it around the steel bar. This time constituted the curing period, during which practically no length changes occurred. Following the curing period, all specimens were placed in a room of 50 per cent humidity. Contraction began immediately and tension began to develop in the concrete against an equal amount of total compression in the steel. Periodic measurements of strain with a Whittemore gage revealed the amount of compression in the steel and consequently also the equal amount of tension in the concrete. At the time of cracking, therefore, the tensile stress was revealed.

An auxiliary feature of the test was the determination of sustained modulus of elasticity of the concrete during the drying period. Free shrinkage of the concretes was measured on companion specimens of the same cross section but without steel reinforcing. It was assumed that the restrained specimens would have contracted the same amount as these free specimens except for the tensile stress caused by the steel. Therefore, the difference in contraction between the concretes with and without steel was assumed to be the amount the concrete was stretched due to the tensile stress. The ratio of the tensile stress (as determined from the steel stress) to the stretch (difference in contraction per unit of length) gave the sustained modulus of elasticity.

An example of the determination of extensibility of concrete is given below. The data are taken from an actual test and indicate the conditions in a concrete containing standard cement and Long Island Sound aggregate just before the concrete cracked at an age of 81 days.

1. Measured contraction of both concrete and steel in central portion of restrained specimen	212 millionths
2. Corresponding stress in steel (212 x 29)	6150 p.s.i.
3. Total compressive force in steel (6150 x 0.73 sq. in.)	4800 pounds
4. Tensile stress in concrete (4800/15 sq. in.)	320 p.s.i.
5. Free shrinkage of concrete as measured on companion specimens of same sectional area	446 millionths
6. Theoretical extension of restrained specimen (446-212)	234 millionths
7. Sustained modulus of elasticity (320/234)	1.4 million p.s.i.

In the above example, several points of interest may be mentioned. First, the restrained contraction was about half of the free shrinkage, indicating that the test condition provides about 50 per cent of complete restraint. Second, the specimen failed at an average stress of somewhat more than 300 p.s.i. Finally, the "sustained" modulus of elasticity was only 1,400,000 p.s.i. The ordinary modulus of elas-

ticity of this concrete as determined in the usual quick-loading test was about 4,000,000 p.s.i., from which it follows that plastic flow accounted for more than half of the observed "stretch."

The dimensions of specimens were more or less arbitrary. As a first trial, the concrete columns were 5 by 5 by 36 inches. The steel rods had a diameter of one inch and were threaded for ten inches near either end. This left well over a foot of the steel bar to be wrapped in the region of the midlength. Strains were measured on opposite sides of the concrete on 10-inch gage lengths. Although previous tests seemed to substantiate the assumption that both concrete and steel changed in length by equal amounts in the central region, further tests are now being made to check this assumption. According to the previous tests, the maximum steel stress to be expected, about 10,000 p.s.i., should be developed within a distance of about five inches from either end. Ten inches was allowed to provide a margin of safety.

No cracking occurred in some of the first specimens, wherein the concrete section was five inches square, during several months of drying. The concrete section was reduced to four inches square, therefore, while the diameter of the steel rod was maintained at one inch. Even then, cracking did not occur for several months in the most resistant concretes.

PRELIMINARY RESULTS OF TESTS

The tests completed thus far have been mainly of an exploratory nature, seeking to perfect the test method. Some of the results are sketched briefly below to show what may be expected, with the reservation that they have not been checked sufficiently to be accepted as final.

Effect of Duration of Moist Curing. One of the disturbing results of ordinary tests of drying shrinkage was that prolonged moist curing did not reduce ultimate shrinkage materially, if at all. But in the cracking test, it was found that the prolonged moist curing helped materially, at least to the extent of delaying the cracking greatly. The following results on concrete containing standard cement and aggregate from Long Island Sound illustrate the effect of curing.

Days of Preliminary Moist Curing	Age at Time of Crack	Days of Drying before Crack
3 da.	19 da.	16 da.
14 da.	81 da.	67 da.
33 da.	(not yet cracked after drying 2 mo.)	

Effect of Type of Aggregate. Early results to compare 3 aggregates indicate that cracking of concrete containing different aggregates occurs about as indicated by relative free shrinkages. For the aggregate exhibiting highest shrinkage, a glacial sand and gravel, cracking occurred at an age of 52 days, after 38 days of drying. For the intermediate aggregate, a sand and gravel from Long Island Sound, cracking occurred at an age of 81 days, after 67 days of drying. For the aggregate of lowest shrinkage, a manufactured dolomite from Virginia, cracking had not yet occurred at an age of 85 days. The cement content and consistency was the same for all aggregates, and the water-cement ratio was allowed to vary.

Effect of Cement Content. Another disturbing result of ordinary shrinkage tests was that lean mixes of concrete appeared to shrink about as much as did rich mixes. In the cracking test, however, a rich concrete cracked in 39 days of drying, as compared with 67 days for a lean concrete. The mixes were 1:4.0 and 1:5.5, respectively, and the water-cement ratios were 0.52 and 0.69. The maximum size of aggregate was $\frac{3}{8}$ inch in either case and the consistencies of both concretes were alike.

Effect of Maximum Size of Aggregate. Early tests showed the effect of maximum size of aggregate on cracking of concrete to be about as expected from the usual shrinkage tests. Up to a maximum size of $\frac{3}{4}$ inch, the larger the aggregate, the less the shrinkage and the greater the resistance to cracking. The difference, however, between aggregates of $\frac{3}{4}$ inch and $\frac{3}{8}$ inch maximum sizes was so small as to suggest that for larger sizes there might be no further benefit, or even a reversal of trend.

Effect of Air-Entraining Agents. The beneficial effect of air-entraining agents, such as beef tallow, on increasing the extensibility of concrete appeared to be more than offset by the greater shrinkage and lower strength. First results indicated much less resistance to cracking for concretes containing 5 to 10 per cent of air.

The foregoing results are but a sample of the information which might be gained from the cracking test. If the test method proves to be satisfactory in continued work, its main advantage may be that it takes account of the behavior of the concrete from the very moment it is cast to the moment when it cracks.

Discussion, to close in the September Supplement, should reach A. C. I. Secretary by July 20.

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Discussion of a paper by Roy W. Carlson:

Attempts to Measure the Cracking Tendency of Concrete*

BY M. O. WITHEY, ROY W. CARLSON, RAYMOND E. DAVIS AND C. A. G. WEYMOUTH

CONVENTION DISCUSSION

M. O. Withey†—What method do you use to determine the incipient cracks? Observation, or did you have a watermark?

Professor Carlson—No special means of detection were necessary because the cracks were so easy to see. One day there might be no crack, and the next day there was an obvious crack.

Raymond E. Davis‡—Your table showed a tensile strength of 320 p.s.i. Would you take that to be the maximum stress that actually existed at the cross section, or would that be the average?

Professor Carlson—Professor Davis has raised a very good point. If you picture the drying of these concrete bars, the drying occurs from the surface, and this particular concrete had dried for 67 days. In that time the drying was by no means believed to be uniform across the section of the concrete. Thus the stress of 320 p.s.i. must, I believe, be taken as an average, and the tensile stress at the surface of the concrete would be expected to be as much as 50 per cent higher.

BY C. A. G. WEYMOUTH**

Mr. Carlson has begun, in this paper, consideration of a phase of concrete shrinkage and cracking that the writer has long felt to be of vital importance, namely the damage done to structures in the drying period before hard set of the cement has been effected. Mr. Carlson

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proposes to take "account of the behavior of the concrete from the very moment it is cast to the moment when it cracks."

The occurance of cracks during the "pre-set" period is so frequent, especially in flat work, it is surprising such little mention has been made of it in concrete literature. Yet positive, or even incipient, cracks produced in this pre-set period are often more dangerous to continuity than those produced by the normal drying out after the cement has become hard set. The writer wishes to offer a few comments to stimulate further discussion on this phase of drying shrinkage.

Shrinkage of Colloidal Materials

Everyone has noticed cracks opening up in fine grain soils upon drying out. Fruit jellies also shrink from the sides of a jelly glass upon exposure to dry air. Clays contain considerable colloidal material that gels like fruit jelly from water solutions when sufficiently concentrated. Cement contains some of this ultra fine material, and when mixed with water, hydration at the surfaces of coarser cement particles produces flocs that are rubbed off into the mixing water and which become colloidal and gel upon the setting of the cement.

Shrinkage of cement mortars and concretes during the pre-set period is of the same nature, but of different degree, as that of moist clay. Clay contains both fine and coarse particles; drying shrinkage results from the decrease in volume of the colloidal element as well as from the effect of surface tension in pulling the coarser solids together as drying proceeds.

Drying Changes in Concrete Before Hardening

When screeding of flat work or rodding and vibration of concrete in forms is completed, all solid particles settle slowly at a uniform rate until the coarser particles become arched and fixed in the mass. Each smaller size will then stratify until it in turn touches other particles. This settling action exposes water at the surface which proceeds to evaporate in the presence of dry air, warm weather, and wind, singly or combined. The structure formed by the settlement of the solids to a rigid condition is rather loose and the smaller particles are still far from consolidated, being held apart by the water films on their surfaces.

As the mixture dries out, the contained water moves through the mass to the drying surfaces and may cause further readjustments. When drying has proceeded beyond the point of particle adjustment, air is drawn into the mass to replace the volumes in the voids formerly filled with water. From this stage on to the final drying or to some point before hard set, surface tension plays its part in the shrinkage of the mass.

Effect of Surface Tension

Surface tension is popularly thought of as if the film in the air-water plane were stretched like a rubber band. While there is no actual tension in this film, it is in a state of stress and the effects of this stress appear as if real surface tension existed. The energy forces involved are always tending to reduce the air-water surface to a minimum. When water readily wets a solid, like cement and aggregate particles, it spreads and thus new air-water surface is created. Here the attraction of water to the solid is greater than the surface-energy effort to reduce the amount of air-water surface. These two opposing forces result in curved water surfaces in all voids and capillaries in concrete where air, water, and solid meet. The meniscus provides less surface than a straight intersection with the surface plus the area upwards along the solid to the same elevation.

When the walls of the capillary are fixed the surface energy tends to produce movement of the water upwards or along the capillary. But when these walls are composed, as in concrete, of the faces of adjacent but independent particles, not only does the water move upwards but also the particles are drawn closer together to reduce the total air-water surface in the system, and this causes shrinkage.

Hence shrinkage due to drying in the pre-set period is from two causes: (1) a loss of water from the cement gel and a thinning of the films separating the particles, (2) a physical pulling together of the solids by surface tension.

Pre-set cracks occur in slabs, in footings on fine sandy soils, in the tops of walls or beams. They should be sealed up by delayed action —rolling, tamping, or trowelling with heavy pressure—just before plasticity disappears. If not sealed throughout their depth these cracks will continue for the life of the building. Incipient cracks become focal points of weakness to crack at a later drying period. Anything done in construction to densify the cement gel by permitting hydration to proceed as far as possible before final working will reduce ultimate shrinkage and improve the strength of concrete.

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Stress Increases in Compressive Steel Under Constant Load Caused by Shrinkage*

By G. A. Maney and M. B. Lagaard†
MEMBERS AMERICAN CONCRETE INSTITUTE

SYNOPSIS

The authors present data from various sources, which have formed the basis of their conclusions that continued deformations in compressive steel of reinforced concrete members under constant loads are caused by shrinkage and not by "plastic flow" or "creep." This applies to the common case in practice where the compressive steel receives its load only through the bond in the concrete and not by direct application at the ends.

They do not contend that "flow" of concrete does not occur under high loads, but present the opinion that a "flow" in bond occurs as well as a flow in compression, thereby causing no additional stress in the reinforcement.

They explain the additional deformation in a concrete column beyond that normally attributed to shrinkage, by an increase in load intensity due to reduction of the effective compression area from warping. This is caused by more rapid drying in the outside elements of the column.

They present evidence to explain the high time deformations on the compressive surface of plain concrete beams under sustained load, on the basis of eccentric shrinkage rather than flow.

What is usually called flow or "creep" in a drying out column under sustained load is identified by them as shrinkage since changes of comparable magnitude are not found to occur in wet loaded columns.

An attempt is made to re-allocate the responsibility for time changes under load, between shrinkage and flow.

Shrinkage and plastic flow as names of factors controlling volume change of concrete have been used more as synonyms than as words with an independent meaning. Since 1930 when considerable research

*Presented 36th Annual Convention American Concrete Institute, Chicago, Feb. 27-29, 1940.

†Chairman and Assistant Professor, respectively, Department of Civil Engineering, Northwestern University, Evanston, Ill.

activity developed in the investigation of volume change, plastic flow has been given equal or more than equal credit with shrinkage as a source of volume change in the concrete of a normal structure under working loads.

Evidence is here presented which indicates the controlling nature of shrinkage as a factor in volume changes under *working* loads and in compressive steel stress under *all* loads.

"Flow" or "creep" is here defined as a volume change due to application of load only. Permanent set or change of shape after release of load is essential to flow of material and rate of flow or amount of set is known to increase rapidly with high intensities of load.

Shrinkage is volume change independent of load and is due to evaporation of colloidally held water from the cement paste of concrete. The amount of shrinkage has been shown to be roughly proportional to water loss and also to the percentage of absolute volume of cement paste to total volume of concrete bound together by this paste.

The existence of the phenomenon of plastic flow in concrete under high loads is, of course, unquestioned. Its importance under working loads is found to be negligible as is also its effect on time increases in compressive steel stress under sustained load.

Expressed in a definite quantitative manner, the following statements are supported not only by the evidence collected by the writers in the Northwestern University School of Engineering laboratories, but by other important data published during recent years:

1. All increases of unit strain and stress in compressive steel of beams or columns under sustained loading are closely identical with the unrestrained shrinkage of an identically made and stored specimen without steel and with no load.

2. The unit strain in compressive steel at any time after loading and drying out may be expressed as $E_c + s$ where E_c is the instantaneous load unit deformation in the concrete and s is the unit shrinkage coefficient of the unrestrained identical specimen without load or steel. The unit stress in the compressive steel at any time may then be expressed as follows: (See Fig. 1)

$$f_s = nf_c + sE_s$$

nf_c = the elastic stress in the concrete times the ratio $\frac{E_s}{E_c}$ and is the common form for the load increment.

sE_s = the unit shrinkage coefficient times the steel modulus and is the stress increment due to sustained loading.

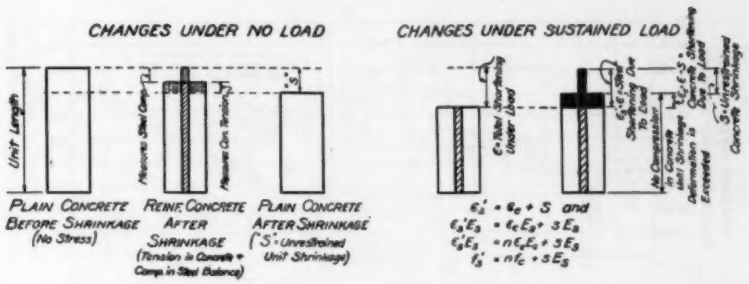

CHANGES UNDER NO LOAD
CHANGES UNDER SUSTAINED LOAD
Unit Length
Plain Concrete Before Shrinkage (No Stress)
Reinf. Concrete After Shrinkage (Tension in Concrete + Comp. in Steel Balance)
Plain Concrete After Shrinkage (S = Unrestrained Unit Shrinkage)
$\epsilon_s' = \epsilon_c + S$ and
$\epsilon_s' E_s = \epsilon_c E_s + s E_s$
$\epsilon_s' E_s = n \epsilon_c E_c + s E_s$
$f_s' = n f_c + s E_s$

FIG. 1—EFFECT OF SHRINKAGE DEFORMATION ON COMPRESSIVE STEEL STRESSES UNDER SUSTAINED LOAD

3. In plain concrete under working loads the time increases of deformation with sustained loading are very largely explained by shrinkage alone.

If the dropping of nf_c from column codes is justified on the basis that under ultimate load conditions the elastic limit of the steel is developed, it also surely seems to be justified in compressive beam steel. Architects and engineers are deeply interested not only in a safe use of reinforced concrete but also in knowing the extent to which time changes may affect the configuration of reinforced concrete columns and flexural members.

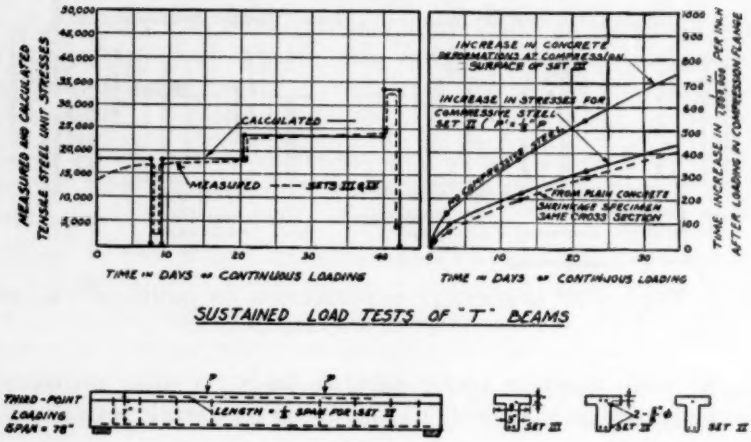
There is little doubt that the near future will offer methods of controlling and minimizing concrete volume changes. The first step in this control should be a very carefully considered analysis of causes of volume change. It is to the end of correct diagnosis of volume change that the evidence of this paper is offered. It is thought that a "question-answer" series would be effective in considering this subject:

1. What kind of control specimen will determine the contribution of shrinkage to time-increases in compressive steel stress under sustained load?

Fig. 1 indicates very clearly that a control specimen identical with the reinforced specimen under load indicates nothing except the amount by which the given steel percentage retards the normal or unrestrained shrinkage of the concrete. Only a plain concrete specimen will indicate how much skrinkage must be overcome by load deformation before the concrete is actually loaded in compression. Fig. 1 also clearly indicates how the expression:

$$f_s = nf_c + sE_s$$

gives the compressive steel unit stress in terms of the usual nf_c which is always true for instantaneous loading and sE_s which is the increment due to shrinkage under sustained loading.


MEASURED AND CALCULATED TENSILE STEEL UNIT STRESSES
CALCULATED
MEASURED --- SETS III & IV
TIME IN DAYS OF CONTINUOUS LOADING
INCREASE IN CONCRETE DEFORMATIONS AT COMPRESSION SURFACE OF SET III
INCREASE IN STRESSES FOR COMPRESSIVE STEEL SET III (P' = 1.4 P)
FROM PLAIN CONCRETE SHRINKAGE SPECIMEN SAME CROSS SECTION
TIME IN DAYS OF CONTINUOUS LOADING
TIME INCREASE IN UNITS PER IN. AFTER LOADING IN COMPRESSION FLANGE
THIRD-POINT LOADING SPAN = 78"
LENGTH = 1/2 SPAN FOR SET III
SET III
SET IV
SET II

SUSTAINED LOAD TESTS OF "T" BEAMS

FIG. 2

2. What evidence indicates that shrinkage alone accounts for practically all stress increases in compressive steel due to sustained loading?

The following test results all indicate that these time increments of compressive steel unit strain are practically identical with the unit shrinkage of plain concrete control specimens of the same concrete cross section and drying out conditions.

(*a*) The Fig. 2 and 3 show results from the beam tests made at Northwestern University in 1929. In Fig. 2 attention is called to the close check between calculated and measured stresses in tensile beam steel. The calculations, which are indicated in the upper left hand diagram, were made by the conventional method, $f_s = \frac{M}{jdA_s}$ which neglects tension in the concrete. In the case of these T-beams, the web stresses were high and after about 10 days the cracking of the concrete below the neutral axis seemed to eliminate practically all of the help of the concrete to the tensile steel.

In the upper right hand corner of Fig. 2 and in the upper half of Fig. 3 increases with time only are shown with measured deformations of compressive concrete and compressive steel. Much larger increases with time are seen for beams with no compressive steel than for beams with compressive steel. The dotted lines in these two diagrams represent the measured shrinkage in an unreinforced specimen of the same cross-section of these beams, which were allowed to dry out under no load and under the same conditions of humidity.

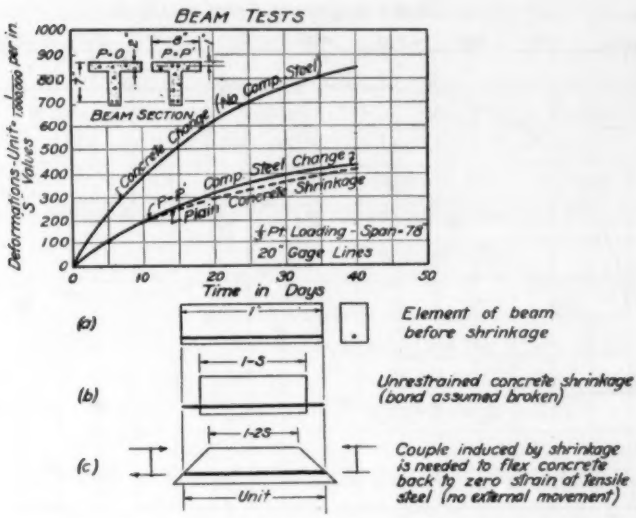

BEAM TESTS
Deformations-Unit = 1/1000000 per in
"S" Values
1000
900
800
700
600
500
400
300
200
100
0
P-0
P-P'
BEAM SECTION
Concrete Change (No Comp. Steel)
P-P' Comp. Steel Change
Plain Concrete Shrinkage
¼ Pt. Loading - Span-78"
20" Gage Lines
0
10
20
30
40
50
Time in Days
(a)
1"
Element of beam before shrinkage
(b)
1-S
Unrestrained concrete shrinkage (bond assumed broken)
(c)
1-2S
Couple induced by shrinkage is needed to flex concrete back to zero strain at tensile steel (no external movement)
Unit

FIG. 3

36"
2"
8"
8"
8"
8"
2"
A
B
4½"
4-½ϕ Def. Bars
½"
½"

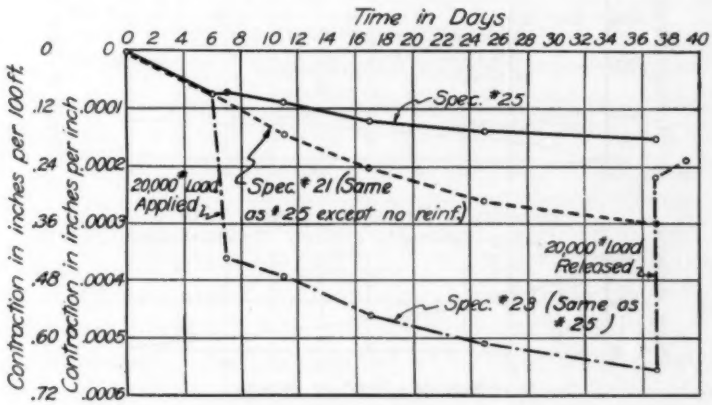

Time in Days
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40
Contraction in inches per 100 ft.
0
.12
.24
.36
.48
.60
.72
Contraction in inches per inch
0
.0001
.0002
.0003
.0004
.0005
.0006
Spec. #25
20,000# Load Applied
Spec. #21 (Same as #25 except no reinf.)
20,000# Load Released
Spec. #23 (Same as #25)

FIG. 4

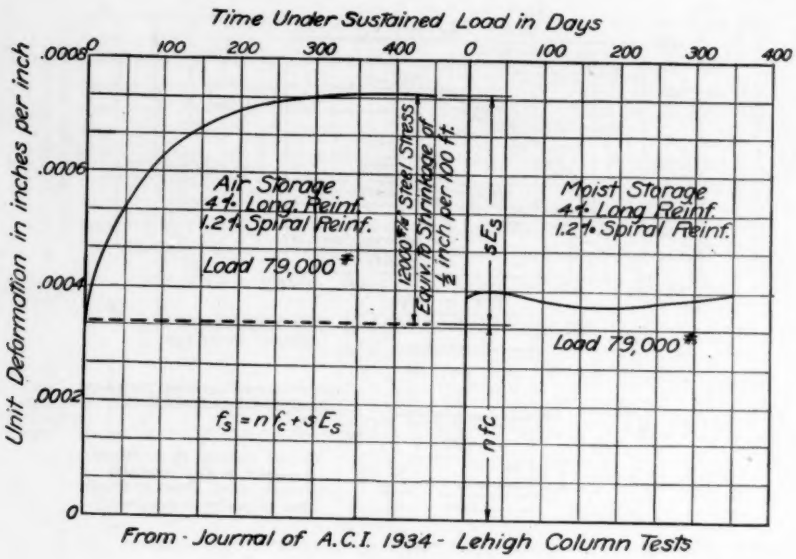

Time Under Sustained Load in Days
0
100
200
300
400
0
100
200
300
400
Unit Deformation in inches per inch
.0008
.0006
.0004
.0002
0
Air Storage
4% Long. Reinf.
1.2% Spiral Reinf.
Load 79,000#
12000 #/" Steel Stress
Equiv. to Shrinkage of
1/2 inch per 100 ft.
sEs
nfc
Moist Storage
4% Long Reinf.
1.2% Spiral Reinf.
Load 79,000#
fs = n fc + s Es
From - Journal of A.C.I. 1934 - Lehigh Column Tests

FIG. 5

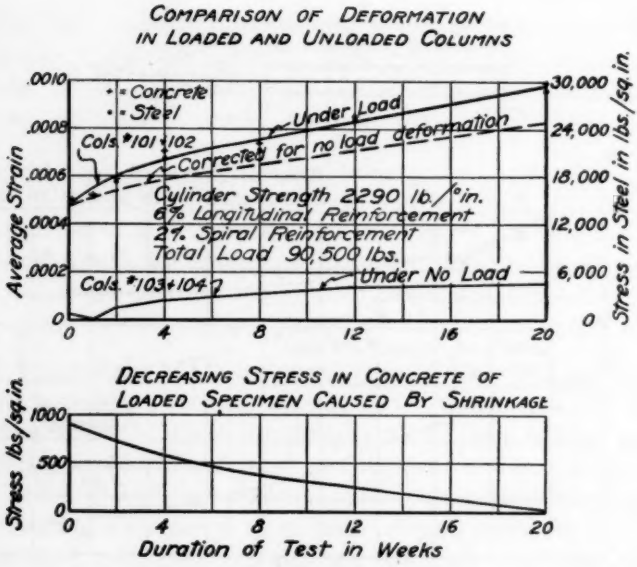

COMPARISON OF DEFORMATION
IN LOADED AND UNLOADED COLUMNS
.0010
.0008
.0006
.0004
.0002
0
Average Strain
• = Concrete
• = Steel
Under Load
Cols. #101+102
Corrected for no load deformation
Cylinder Strength 2290 lb./"in.
6% Longitudinal Reinforcement
2% Spiral Reinforcement
Total Load 90,500 lbs.
Cols. #103+104
Under No Load
0
4
8
12
16
20
30,000
24,000
18,000
12,000
6,000
0
Stress in Steel in lbs./sq.in.
DECREASING STRESS IN CONCRETE OF
LOADED SPECIMEN CAUSED BY SHRINKAGE
Stress lbs./sq.in.
1000
500
0
0
4
8
12
16
20
Duration of Test in Weeks

FIG. 6

Attention is called to the close check between the time increases in the compressive steel and the shrinkage of the control specimens.

These T-beams all have a span of 78 in. with one-third point loading. All strains were measured with 20-in. Berry type strain gages on gage lines at the center of the span and between the loads.

(*b*) Fig. 4 shows the result of a recent test at Northwestern University on columns. It should be noted that specimens 23 and 25 are identical, each having four ½-in. round deformed bars for vertical reinforcement and that specimen 21 has the same length and diameter but has no reinforcing.

It is very clear from this figure that the time increases of the loaded specimen with steel reinforcement (No. 23) are the same as those of the unloaded specimen with no reinforcement (No. 21).

In other words, the time increases in compressive steel strain in the loaded reinforced concrete specimen are identical with the unrestrained shrinkage of the concrete specimen and have no particular relation to the time changes of specimen 25 which is an identical specimen under no load.

The tests described above are based on the average of the two center 8-in. gage lines.

(*c*) The A. C. I. column investigation at Illinois and Lehigh Universities shows compressive steel increments under sustained loading of from 6000 to 25000 p.s.i. (see Fig. 5 and 6).

Fig. 5 is taken from page 304, January, 1932 A. C. I. JOURNAL and represents the average of two columns (a) and (b). These were selected as fairly typical.

Fig. 6 was taken from Fig. 43 of the second progress report on the column investigation made at Lehigh University.

These sE_s increments correspond to a shrinkage range of from ¼ in. to 1 in. per 100 ft. (see Fig. 7). This is a normal range of unrestrained shrinkage for the concrete of these columns.

(*d*) Fig. 5 indicates the average results of two pairs of specimens from the A. C. I. investigation which were typical. Although identical and loaded the same the change in the wet stored specimen is negligible while those loaded under dry conditions showed an increment of compressive steel unit stress (12000 p.s.i.) corresponding to a normal shrinkage over this period of ½ in. in 100 ft. We are forced to conclude that the drying out of the dry cured specimen which allowed shrinkage was responsible if no appreciable change took place in an identical specimen which was not allowed to shrink.

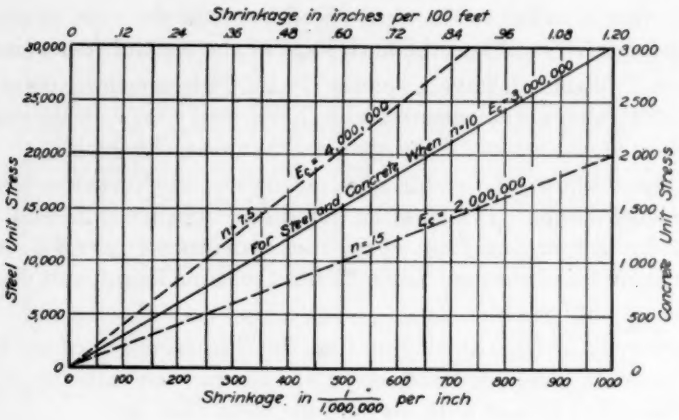


FIG. 7

3. What evidence indicates that plastic flow does not affect stresses in compressive steel when load must get to this steel through bond in the concrete?

If we define plastic flow as identical with the flow or permanent set caused in concrete under high load intensities near the ultimate, Fig. 8 indicates very clearly how a large permanent set in the concrete after release of load does not change the compressive steel stress which was induced in the steel by shrinkage before the high loads causing flow of concrete were applied. In this experiment which was made in 1929 and duplicated recently at Northwestern University, a large permanent set was measured in the concrete after the load causing it was released, while no appreciable change was measured in the compressive steel. The steel under compressive stress due to shrinkage lags behind that shrinkage in unit value because of a 20-in. steel gage line on a specimen 24 in. long. Since the stress in the steel is induced through bond or grip of the concrete, the average steel stress over a 20-in. gage line was measured rather than the maximum steel stress near the mid-length.

4. What evidence from the A. C. I. column investigation tends to eliminate plastic flow as a factor in sustained load increases of compressive steel unit stress?

In Fig. 6 a pair of columns with 6 per cent compressive steel and dry air load conditions is analyzed to show how rapidly the intensity of unit stress on the concrete drops off as the compressive steel gradually takes on more and more of the total load. One cannot visualize plastic flow due to intensity of load continuing under conditions of

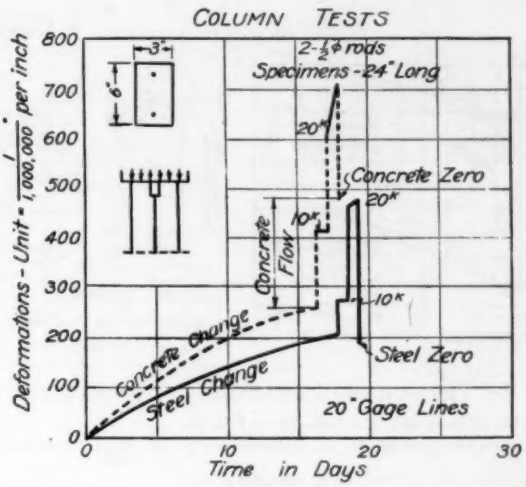


FIG. 8

rapidly decreasing load intensity. This dropping off of unit loading intensities is typical of all concrete in which compressive steel is embedded if loaded under dry air conditions.

5. What evidence is available to definitely differentiate from shrinkage the phenomenon referred to as "creep" or flow of concrete caused by "seepage" of "colloidal water" due to load?

The writers know of no evidence to indicate that concrete under ordinary working loads will dry out or lose moisture content more rapidly than identical specimens under no load and the same drying out conditions. Our tests so far have indicated a very slight increase in water loss by weight from the unloaded specimens.

6. What evidence indicates that under ordinary working stresses the increases in unreinforced concrete deformation due to sustained load are explained entirely by shrinkage?

Typical data from the plastic flow investigation conducted at the University of California (see Fig. 9) indicate how the warping of the cross section due to more rapid drying and shrinking of concrete near the surface, with the consequent increase in elastic deformation, can explain the small differences between the time change of loaded and unloaded specimens. The increase in deformation of the loaded specimen during the first 20 days is a result of the change from the uniform stress of the specimen of Fig. 9, Case 4, to the distribution of Fig. 9, Cases 1 or 2, after the more rapid drying out of the outer elements of the specimen. The doubling of the elastic deformation with

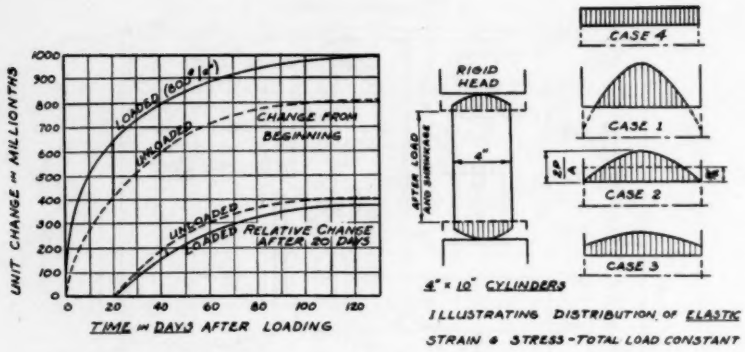


FIG. 9—TIME-CHANGE ANALYSIS OF TYPICAL CYLINDERS

From data reported on page 364, A.S.T.M. Proc., Vol. 34.
27 Day Moist Curing followed by 100° F. Temperature and 25 per cent relative humidity.
CONCLUSIONS—
(A) Time-change differences explained by warping of ends
(B) All change due to shrinkage

an E_c of about 4,000,000 will explain all the early increase of the loaded specimen deformation. After this time a slight tendency toward reducing this difference is noted as the inner shrinkage tends to approach the outer shrinkage and the stress distribution tends to resemble that of Fig. 9, Case 3.

7. What experimental evidence of warping is available?

The data of Fig. 10 (Northwestern University, 1929) show the result of measurements of deformation due to shrinkage across the diameter of an 8 x 32 in. specimen, with a special yoke micrometer. After 20 days a difference of 150 parts in a million is measured between the outside and center shrinkages. With a 4,000,000 concrete modulus this would result in a stress of about 300 p.s.i. of tension on the outer elements and equal unit compression on the central elements of the cross section. This assumes a paraboloid of revolution for this distribution of deformation throughout the cross section. (See Fig. 9, Case 2).

8. What accounts for the experimental evidence that sustained load deformation on the compressive surface of a beam reinforced with tensile steel but not *reinforced with compressive steel is always much more than the unrestrained unit shrinkage of an identical plain concrete specimen?*

Fig. 3 gives a typical case taken from the 1929 investigation at Northwestern University. Below the diagram showing the increase with time is indicated an approximate idea of the distortions involved

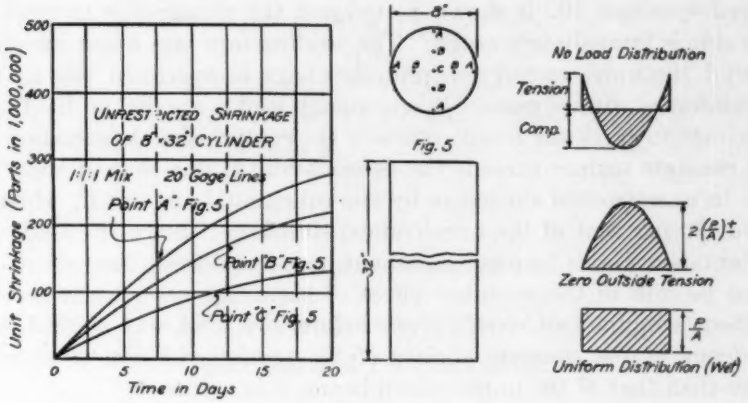


FIG. 10

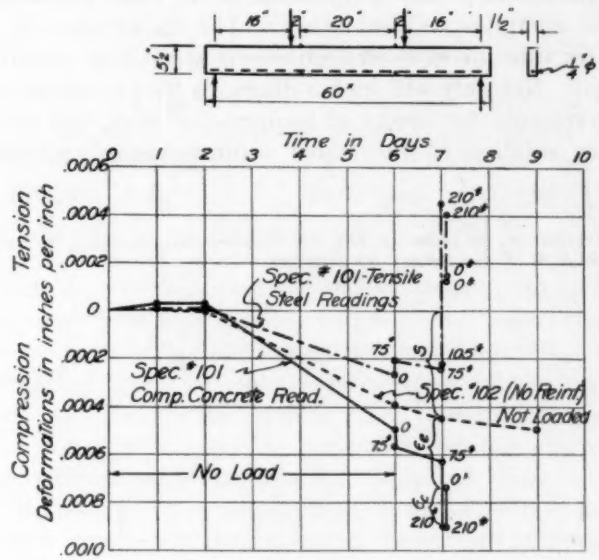


FIG. 11

in the eccentric application of shrinkage to beam section bending with steel on only one side. Fig. 11 shows the results of a recent test at Northwestern University. The two specimens tested are identical in shape, concrete and drying out conditions. Specimen 101 has tensile beam steel and is loaded after several days of drying out with a short application of load. Even before load the warping of the beam due to eccentric shrinkage is evident. The average shrinkage of the unrein-

forced specimen 102 is shown dotted and the change due to steel on one side is immediately noted. The steel surface lags about as much behind the unrestrained concrete shrinkage of specimen 102 as the unreinforced surface gains. When enough load is applied on the beam specimen to crack the tensile concrete the compressive deformation on the concrete surface exceeds the deformation E_c due to load and "S" due to unrestrained shrinkage by the substantial amount E_s which is about 70 per cent of the unrestrained shrinkage. Since the load was instantaneous, this increase E_s cannot be called flow or time effect but must be due to the eccentric effect of shrinkage on a beam section without compressive steel. Even before any load was applied, the tendency of the concrete surface of the reinforced beam to shorten more than that of the unrestrained beam, is in evidence.

CONCLUSION

This evidence all points to shrinkage as the principal factor in time changes of compressive steel stress. The importance of a correct diagnosis for time increases in compressive steel stress cannot be overemphasized. Not only will such a diagnosis lead to more satisfactory code requirements for design of compressive steel, but the eventual satisfactory solution of the whole volume-change problem will be served.

Discussion, to close in the September Supplement, should reach A. C. I. Secretary by July 20.

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Discussion of a paper by G. A. Maney and M. B. Lagaard

Stress Increases in Compressive Steel Under Constant Load Caused by Shrinkage*

BY F. E. RICHART, FREDERICK G. ANDERSON, GEORGE C. ERNST, H. J. GILKEY, A. U. THEUER AND AUTHORS' CLOSURE

CONVENTION DISCUSSION

F. E. Richart†—I will admit that the last statement has me somewhat puzzled because in numerous experiments I found what I thought was plastic flow. In the first place, I will refer to some of the American Concrete Institute column tests, made both at Lehigh University and at the University of Illinois. We measured the shrinkage in columns which were held under sustained load and in columns which were not loaded. We measured the elastic strain or the strain which took place when load was applied, and what we reported as plastic flow had this elastic strain eliminated from consideration. The readings we obtained from the unloaded specimens which were subject only to shrinkage, and the loaded ones which were subject to shrinkage and plastic flow, could then be compared, having eliminated the elastic strain by actual measurement. Now the thing that puzzles me is that we found in many cases in the loaded specimen an increase in strain corresponding to the change in stress from an original value of, say, 10,000 p.s.i. in the steel due to elastic strain, to as high as 30,000 p.s.i. in the steel after a year under load, a gain of 20,000 p.s.i. steel stress. Again in the companion specimen not under load we had gains of four or five thousand pounds; so there is a difference between 5,000 and 20,000 p.s.i. I know it is not exact to speak of measured stresses where we actually measure strain; but it is easier to think in terms of stress in steel.

*JOURNAL, Amer. Concrete Inst., June, 1940; *Proceedings* Vol. 36, p. 541.
†Research Professor of Engineering Materials, University of Illinois, Urbana.

Now Prof. Maney has mentioned the fact that there can be a differential shrinkage, more shrinkage at the outer surface of the specimen than the inside. That may be true. At the same time we had over a hundred columns under sustained loading. About thirty of these tests were continued five or six years. One would think that after five or six years indoors, they would arrive at equilibrium; that the shrinkage that was going to take place in the interior of the column would have had time to take place and that the differential shrinkage which might exist in the early stages of drying would have disappeared.

Another item Prof. Maney has mentioned is that in his test the load was brought to the steel entirely through bond; in most of our column tests that was not the case, the end of the column was made perfectly plane so that the load was applied to the flush end of the steel bars. However, I made a series of column tests at that time in which the bars were stopped about half an inch from the end of the column and the concrete was cut away so that there would be no bearing between the end of the bar and the bearing plate. The stresses induced in these columns by flow and shrinkage during a year of sustained loading was just as great as in the columns in which the bars were directly on a steel plate.

I might add that there is another action in addition to elastic strain, flow and shrinkage, and that is a change in the modulus of elasticity of the concrete over a long period. That modulus generally increases, an effect which would decrease rather than increase the elastic strain.

In conclusion, it should be stated that tensile plastic flow or stretching, under sustained tension, has been found by several investigators. That is an effect that cannot be explained as shrinkage.

Frederick G. Anderson†—It seems to me little has been said about the effect of ties or spirals. In the small specimens illustrated there were neither ties nor restraining steel. Is that left out of consideration in the effect on the vertical bars?

Professor Maney—We always found it was not affected by spiral steel or ties until the concrete had been subjected to ultimate or failing loads. For all practical purposes, that spiral might as well not be there. Professor Richart spoke of the comparison of the loaded and unloaded specimens. The unloaded specimens had the same steel in them as the loaded specimens; what we had there was a restraint of the shrinkage by the steel; we did not have unrestrained shrinkage. As I look over the data in the investigation, I can see that the total

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increase of the steel stress throughout corresponds to what I would call a reasonable shrinkage in those specimens.

BY GEORGE C. ERNST‡ AND H. J. GILKEY**

Plastic flow, shrinkage, and tensile cracking (caused by load or shrinkage) with subsequent tension release are factors which merit a great deal of experimental and analytical investigation. A large quantity of work has been done on flow and shrinkage while tensile cracking is receiving more recent attention. Much remains to be done, however, in coordinating and presenting the existing material in a simplified form such that the relative magnitudes will be apparent and readily utilized for specific structural members, types of loading, and conditions of exposure.

The paper by Professors Maney and Lagaard provides much food for thought concerning the relative responsibilities of flow and shrinkage under *working* conditions. Since flow, shrinkage, and tensile cracking are so closely related, the complete solution to the problem will not be achieved until the role of tensile cracking with subsequent release upon the steel has been explored and correlated with flow and shrinkage.

Sustained loading tests on twenty 3 by 6 in. by 10 ft. beams reinforced with varying percentages of steel and subjected to a wide range of steel and concrete stresses were conducted at Iowa State College and reported by the writers in the Highway Research Board Proceedings, Vol. 15, 1935, p. 81. The tests are related to much of the material presented by Professors Maney and Lagaard, particularly to question 8, p. 550. Since the results of the writers' tests are readily available, only a summary of the findings that appear to be more or less related to question 8 will be mentioned. All figures and page numbers refer to the report in the Highway Research Board Proceedings mentioned above.

1. Slender members, of the type tested, respond quite readily to humidity changes which occur in buildings under normal conditions. Such activity occurred following an initial drying period of from 60 to 160 days in an unloaded condition. (Fig. 7, 8, 9, 10, 13, p. 93 to 96. Also see Summary, Items 2 to 12.)

2. Warping caused by shrinkage of unsymmetrically reinforced beams is quite appreciable. Beams 12 and 13 may be compared with 16, 18, 32, (Fig. 7, 9, p. 93, 95). Also see Summary, Item 3, p. 107.

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3. The relative location of the tensile steel has marked effect in the degree of warping. Compare series B and C (Fig. 14, p. 102).

4. Compression steel of an amount equal to the tensile steel definitely reduces the deflection due to warping. Beams 6 and 10 vs. 5 and 9, 2 vs. 1. (Fig. 7, 9, p. 93, 95). Also see Summary, Item 16, p. 108.

5. The relative effects of tension release, temperature, and humidity should be carefully considered in slender members. (Fig. 13, p. 101, Fig. 9 and 10, p. 95, 96). Also see Summary, Item 9, p. 108.

6. Under high working stresses, the additional deflections and strains due to plastic flow and tension release may become quite appreciable and appear to be of a greater magnitude than those due to shrinkage. (Fig. 7, 8, 9 and 10, p. 93-96).

Other data on shrinkage and plastic flow of plain concrete were secured by one of the writers during 1928 and 1929 as part of the auxiliary test program of the Engineering Foundation Arch Dam Investigation of the model of the Stevenson Creek Dam.* Sustained loading tests were conducted on 4 by 14 in. compressive cylinders, (using the technique of R. E. Davis of the University of California) and on 3 by 3 by 40 in. beams. Loads were 25 and 50 per cent, respectively, of the ultimate at the time the loads were initially applied at ages of 3, 7 and 28 days of standard curing by immersion for compression, and at 7 days, 28 days and 3 months for flexure. Some of the specimens were kept immersed during the entire period of sustained load and others were located in the dry air of the laboratory so that both shrinkage and plastic flow were present. There were unloaded control specimens and data on shrinkage and swelling were secured from companion 3 by 3 by 40 in. beams both immersed and in air, on which regular observations were taken for change in length (volume change).

Some of the plotted data on the flexural flow tests are shown on Fig. 13 (number is missing) at the top of p. 705 *Proceedings* Amer. Concrete Inst., Vol. 32 (1936) or on the same page of the May-June 1936 JOURNAL. Compressive flow data are tabulated on p. 706 and are shown graphically on p. 707 (Fig. 11). (See p. 784 of the *Proceedings*, or 1936 Supplement to the JOURNAL, for corrections of "make-up" blunders on legends and figure numbers).

A few of the conclusions from these tests are in excellent accord with some of the authors' observations.

*The Engineering Foundation Arch Dam Investigation, Vol. II (May 1934), "Tests of Models of Arch Dams and Auxiliary Concrete Tests Conducted by the Bureau of Reclamation at the University of Colorado," by J. L. Savage, Ivan E. Houk, H. J. Gilkey and Fredrik Vogt. (Published and distributed by The Engineering Foundation, 29 West 39th St., New York), pp. 518, 519. (Tables 64 and 65) and other portions.

1. The flow of concrete under load in dry air is two or more times as great as that for concrete that is kept wet during the loaded period. This was true for both compression and flexure and apparently so for tension. Flow under water appears virtually to cease after a short time.

2. For unreinforced concrete subjected to sustained flexural load, the writers are not prepared to state whether or not shrinkage effects the amount of flow deflection. Shrinkage was present in the plain concrete beams tested.

3. There is a recovery approximately equal to the elastic portion of the strain upon the removal of load. This occurs at once. There appears to be a slight additional recovery that extends over a period of several days.

An interesting illustration of the relative importance that drying shrinkage may assume was noted in connection with some unpublished reconnaissance tests on tensile flow under sustained loading. These were 3 by 18 in. cylinders moist cured by immersion up to the time of placing in dry laboratory air under sustained tensile loads, applied through springs, the loads being 25 per cent of the tensile strength of the concrete at time of loading. The drying shrinkage exceeded the elongation from plastic flow. There was, therefore, a net shortening for both the loaded specimens and the unloaded controls, the difference between the two representing the plastic elongation they sustained from load.

If the drying shrinkage dominates the sustained loading picture to the extent that this paper implies, then it appears that the question of whether or not there is for concrete, an end-point to plastic deformation under a sustained load which is short of ultimate collapse, is rather decisively answered in the affirmative. The deformation may be expected to cease when drying out of the concrete attains equilibrium with its surrounding atmosphere. As was mentioned earlier, relatively small air-exposed specimens regularly increase in weight during the moister parts of the year and lose weight when the air is drier. There are corresponding volume changes.

The results recorded in this paper should certainly be tied in and correlated with the large volume of pertinent recorded data from such notable researches as those of R. E. Davis and of J. R. Shank. Incidentally Professor Shank's work which was published both as a bulletin of the Ohio State University Engineering Experiment Station and as a paper of the American Concrete Institute (Vol. 32, p. 149, JOURNAL,

Nov.-Dec. 1935) supplies an excellent list of references on work in this field.

BY A. U. THEUER*

After several readings of this interesting paper a degree of uncertainty still remains in the mind of the writer regarding the authors' exact viewpoint. Since the authors have chosen the question-answer method of presenting their views this writer desires to present these additional questions for the authors to consider.

(1) In the definition of creep as "a volume change due to the application of load only" do the authors signify by the term "load" an external load directly applied only?

(2) Do the authors imply in their definition of shrinkage that, excluding the effects of external load, shrinkage is a function of the single parameter "loss of colloidally held water"? Just how do they distinguish experimentally between colloidally and mechanically held water? And further, just what experimental evidence would they cite to show that a volume change will not occur with the loss of moisture, as determined by weight changes, regardless of kind of water or causes?

(3) Do the authors have any evidence or hypothesis to offer that the amount or kind (elastic, permanent, residual, etc.) of deformation resulting from a given internal stress will vary with the manner in which that stress is produced?

(4) If the answer to this question is negative an additional question arises. Would not creep enter as an item in the case of the warping of the cylinder of 1929? (A stress of 300 p.s.i. in tension is certainly in the region of high stresses if not "high loads" where plastic flow is "unquestioned.")

The writer recognizes that the lumping of all volume changes not due directly to an externally applied load (or something that can be independently measured in pounds) as shrinkage, has advantages for certain practical purposes. It should be pointed out, however, as the following experiments cited will perhaps more clearly indicate, that shrinkage used in the above stated sense does not have a single or definite meaning, as is pointed out by Dr. Busch in his compendium on the Plastic Behavior of Concrete.

Using 3- by 8-in. cylindrical specimens of two different concretes and a drying temperature of 80°C, the shrinkage per gram loss of weight for a specimen cured under water for 7 days before drying was found, in tests by the writer, to be but 70 per cent of that for two

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other similar specimens similarly cured respectively for 28 days and 6 months. The results of other experiments using different drying temperatures indicate that not only loss of weight but rate of loss of weight is a factor in shrinkage. Earlier experiments by Graf[1] indicate this same phenomenon. Whether or not one wishes as a matter of choice to admit that creep is an accompaniment of shrinkage, this writer cannot visualize for any but an ideal case (infinitely small dimensions or extremely slow rates of loss of moisture) how shrinkage can take place without accompanying stress.

In conclusion, reference is made to a recent article by Sartorius-Thalborn[2] who, on the basis of the data provided by the A. C. I. column investigation (in part the same as used in the author's analysis), analyzes the stresses induced in the steel reinforcement of concrete columns subjected to shrinkage, considering plastic flow as an integral element. He concludes that shrinkage cannot take place without the appearance of creep in concrete whose free deformation is in any way restrained.

AUTHORS' CLOSURE

The authors are gratified to receive the pertinent and well considered discussions of Professors Ernst and Gilkey and of Mr. Theuer.

The six points regarding warping of light flexural members based on tests in the Highway Research Board Proceedings all are in direct agreement with the authors' answers to their questions 7 and 8. The whole subject of warping of concrete members caused by the unequal rates of drying out of the various elements of the cross-section is extremely important in any attempt to segregate the relative influences of shrinkage and plastic flow on time volume change in concrete.

In their discussion of the Auxiliary Test Program of the Engineering Foundation Arch Dam Investigation, Professors Ernst and Gilkey verify the fact that there is greater time yield for dry loaded test specimens than for the companion wet loaded specimens. In the opinion of the authors the attributing of this increase in time yield to plastic flow is one of the two or three important misinterpretations of experimental results which have been common to shrinkage and plastic flow investigations. Question 6 raised by the authors together with its answer and the data shown in Fig. 9 explain very clearly how this increase in yield is an elastic one due to a gradual concentration of compressive resistance on the inner elements of a section which accompanies a more rapid drying out of the outer elements of the section.

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[1]Otto Graf, Deutscher Ausschuss für Eisenbeton, Vol. 74. (1933)
[2]Sartorius-Thalborn, Beton u. Eisen, Vol. 37, No. 13, p. 219. (1930)

The explanation of all the time yield for the very interesting 3 x 18 in. sustained tension loaded specimens must also be shrinkage. The outer elements near the surface dry out much more rapidly and finally shrink much more than the inner elements of the specimen. Assuming the common law of a plane section before stress remaining a plane after stress, this unequal shrinkage causes an initial tension in outer elements and an initial compression in the inner elements of unloaded specimen. When it *dries* out under load the stress distribution gradually changes from a nearly uniform distribution in the still moist cylinder under early load to the distribution indicated in Fig. 10 of the authors' paper (page 551, Vol. 36). As this change takes place there is an *elastic* increase in deformation which must be charged to shrinkage but which has always been called "plastic flow". Whether the member drying out under load is in tension or compression the result is the same—an increase in deformation in the direction of the load which is *not* caused by "flow" but which is an increase in strain due to increase in unit stress and where this increase in unit stress can only be caused by redistribution of resistance across the section on account of unequal drying out and shrinkage. If this is "flow", which is defined as time yield due to continued load and which depends for its unit amount on intensity of unit load and which is supposed to increase at a greater rate than stress intensity—several questions become very difficult to answer. One of the most difficult questions to answer and one which has been hinted at by Professors Ernst and Gilkey is: "Why is this so-called *flow* coincident with and always increasing with shrinkage and why does it *end* when shrinkage ends?" The only answer is that practically all of what has been called flow is shrinkage and can be easily explained by a knowledge of the warping of a member which increases with its drying out.

A second and equally common misinterpretation of test data is, in the opinion of the authors, the so-called "seepage" of water which is caused by intensity of load on a concrete specimen and which supposedly accounts for the fact that specimens under load shorten more than specimens under no load. This makes flow a function of the seepage or squeezing out of moisture caused by this load.

It is generally admitted that shrinkage results from a flow of moisture from a specimen which can be best defined as due to a difference of humidity potential between the drying out specimen and the surrounding air into which this moisture flows. Any change of volume due to flow of moisture from the specimen which is independent of load must be called shrinkage. If loading will squeeze water from a specimen then change of shape or decrease in volume may be attributed to

flow. Briefly, plastic flow depends on nothing but load intensity whereas shrinkage is due entirely to a loss of moisture which is independent of the intensity of load on the test specimen.

Question 5 by the authors mentioned certain tests which compared the loss of weight due to evaporated moisture between loaded and unloaded specimens. This point is so important that the authors carefully tested six identical specimens for a long period for loss of weight due to loss of moisture. Three of these were under a comparatively high working load of 1,000 p.s.i. Great care was taken to keep the other three specimens under no load in exactly the same location in the room, at the same height from the floor and with exactly the same surfaces exposed for purposes of evaporation. (See Fig. 12 for the results of this experiment.)

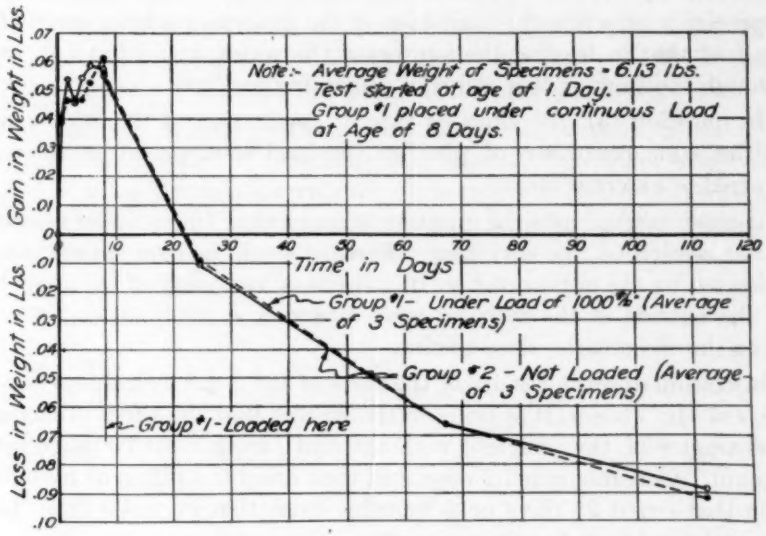

FIG. 12—A COMPARISON OF LOSS OF WEIGHT BETWEEN LOADED AND UNLOADED PLAIN CONCRETE SPECIMENS DUE TO DRYING OUT OF MOISTURE

For all practical purposes there is no difference in the loss of moisture between these two sets of specimens. We are satisfied as a result of this and previous tests that water is not squeezed from the specimens under load because of this load, and that there is no such thing as colloidal "seepage."

The discussion of Dr. Theuer brings out a number of questions and points of interest, and the authors are pleased to have this contribution to the subject.

In reply to question (1), the definition of "creep" as used in the text is a volume change due to an external load directly or indirectly applied.

In question (2), the tests of Professor Maney, published in *Engineering and Contracting* in April, 1931, indicate that water that is held mechanically in a piece of concrete may be released without affecting the shrinkage to any appreciable extent. However, there does not appear to be any definite point where one action ceases and the other starts. It is quite probable that water may be held mechanically in the interior of a mass of concrete at the same time that colloidally held water is being evaporated from the surface. Likewise, water absorbed by certain types of aggregates may evaporate without appreciably affecting the shrinkage of the mass as a whole, except for the fact that in leaving the aggregate the water would have to pass through the surrounding colloidal cementing medium.

In question (3), the effect of rate of application of the load would be the same regardless of whether the load were produced through internal or external means.

If creep entered into the question of warping of the cylinder referred to in question (4) the shrinkage difference would be even greater than indicated by the curves of Fig. 10. If the extensibility of the concrete on the outside of the cylinder were exceeded, a readjustment would occur throughout the cross-section.

In considering the results of the tests of the 3 x 8 cylindrical specimens of Dr. Theuer, it appears to the writers that the effect of change of character of the concrete with age and curing must be taken into account. Concrete cured 7 days and then dried is a different material from that cured 28 days or 6 months, even though made from the same mix and tested at the same age.

In summary it might be emphasized again that so-called "plastic flow" is reported in concrete cured under drying conditions and is not evident in concrete that is kept moist, in spite of the fact that the moisture in the concrete would tend to lower its strength in compression* and thereby place the existing applied stress nearer the ultimate strength. If the material actually flows under working loads, it would be expected to flow more in the wet concrete than in the dry.

*"Saturation reduces strength of concrete," *Engineering News-Record*, 1918, by M. B. Lagaard.

Conclusion

It becomes evident that a reinterpretation of all available test data together with additional tests should be made in the light of two very important experimental facts:

1. Warping exists in all concrete members during the process of drying out. Evidently much greater ultimate unit shrinkage is obtained by the outer than by the inner elements of a member and the effect of size on shrinkage is important. Briefly, in a member losing water over a long period of time such as the inner elements of a large concrete member, shrinkage is much less than for small members which dry out rapidly and also is much less for the outer elements of these large members. The lack of recognition for the important roll of warping in the interpretation of test data has been largely responsible for much confusion as to the relative importance of shrinkage to plastic flow.

2. Loss in weight due to moisture evaporation is a function of a difference in humidity potential only and is not affected by the load. This fact merely reinforces the statements under conclusion 1.

The authors extend their appreciation to the writers of the discussions that appear in this issue and also to the others who have discussed these conclusions with the authors. It is hoped that interest will continue in this subject until enough data have been obtained from different sources to clear up all points that are now in doubt.

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Permeability, Acid, and Absorption Tests of Mortars Used in Dry Tamped Silo Staves*

By C. A. Hughes†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

Data are presented on the permeability, acid resistance, and absorptions of specimens cut from concrete silo staves of widely varying properties.

A method of test for relative acid resistance was developed using 0.16N lactic and acetic acid solution. Data from companion tests using phosphoric acid solutions, molasses solutions, corn juice and silage juice indicate that the lactic and acetic acid solution may be used to determine qualitatively the relative resistance of mortars to the above solutions. Quantitatively the relative resistance depends both on the solution and the condition of exposure. For these tests, the resistance of mortars to corrosion increased with increase in the flexural strength and with increase in the cement content but no "critical" strength or cement content was apparent.

The absorption tests showed clearly that the 24 hour absorption was inferior to the five minute absorption, or the ratio of five minute absorption to the 24 hour absorption as a criterion of mortar quality.

A few data on hydrostatic heads in silos indicate the need for more information on the exposure conditions existing in a silo.

INTRODUCTION

The chief reason for the formation of Committee 714, Recommended Practice for the Construction of Concrete Silos was an uneasiness in the minds of both users and manufacturers as to the effect of so-called "acid action" on the life of concrete stave silos conforming to the A. C. I. 1926 specifications.

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*Presented 36th Annual Convention American Concrete Institute, Chicago, Feb. 27-29, 1940.

†Associate Professor of Structural Engineering, Engineering Experiment Station, University of Minnesota. Professor Hughes is chairman A. C. I. Committee 714.

Sugars, lactic and acetic acids, and carbon dioxide in solution are known to exist at some stage in the formation of silages, and the effects of the interaction of these solutions with the concrete portions of the silos have been observed for many years. The increasing use of forages artificially preserved by the addition in considerable volume of relative strong saccharine and acid solutions served to focus attention on "acid action".

The investigation on which this paper is based is one of several started by members of Committee 714 after the first meeting of the committee in December 1937. Because the field covered by this investigation was largely unexplored when these tests were started, greater emphasis was placed upon the inclusion of what appeared to be the most important variables than on the use of sufficient specimens and repetitions firmly to establish relationships.

Three properties of the staves other than the transverse strength have been investigated. These are the permeability, the resistance to acid, and the absorption. The staves, and data on proportions, transverse strengths, etc. were supplied by Dalton G. Miller, member of Committee 714. A wide range of transverse strengths, conditions of manufacturing, and materials was covered by these specimens.

Because the proper interpretation of laboratory tests depends upon the knowledge of field conditions, some data on the hydrostatic heads in silos are included.

PERMEABILITY TESTS

Purpose

The permeability tests were made because it was believed at that time that hydrostatic heads against silo walls did exist rather commonly, and also, because an air-tight wall favors the preservation of silage in the neighborhood of the wall, that it might be desirable to place some limits on the permeability of staves.

Scope

Permeability tests were made on 23 plant staves and 35 experimental staves. The plant staves represented, at the time they were taken, the commercial product of 9 plants. The experimental staves include as variables the number of blows per tamper (all made at one plant) and the number of staves per sack of cement (made at 3 plants.

Apparatus and Procedure

The essential details of the apparatus are shown in Fig. 1.

In the early tests, those on the plant staves and on a few of the experimental staves, the pressure was 10 p.s.i. corresponding to a hydrostatic head of 23 ft. This pressure was obtained by use of compressed air acting in the standpipes. Due to the high permeability

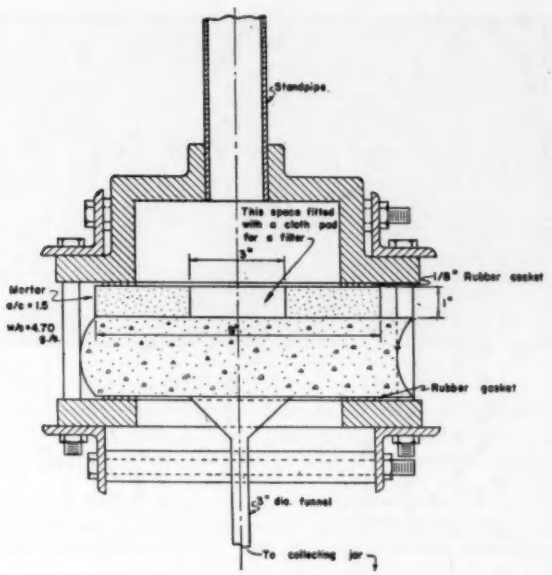

FIG. 1—APPARATUS AND SPECIMEN FOR PERMEABILITY TESTS

of some of the specimens difficulty was experienced in maintaining a constant pressure. Consequently the apparatus was rearranged to provide a constant head of 15 feet of water.

The outflow, that is water passing through the specimen, was collected by a 3-in. diameter funnel held against the lower face of the specimen and leading into a stoppered flask. The extreme permeability of some of the specimens made it necessary to use a very short test period. The rate of outflow at 15 minutes expressed in cubic feet per square foot per hour was used as a criterion of permeability.

In Fig. 2, the rates of outflow are plotted against the transverse strengths and the numbers of staves per sack. To avoid overlapping of the plotted points, data for a number of staves having low permeabilities have not been shown. The limits on average transverse strength of the 1926 A. C. I. specification and of the specification proposed by Committee 714 are shown on Fig. 2 also.

It will be seen from the left panel of Fig. 2 that staves of this group having transverse strengths in excess of 120 lb. per inch of width* had low or zero permeabilities. Staves having strengths of 100 pounds per linear inch or less may have high permeabilities, though down to

*In the silo industry, the maximum load applied at midspan of a 24-in. span and expressed in pounds per inch of width has been used as the measure of transverse strength of the stave.

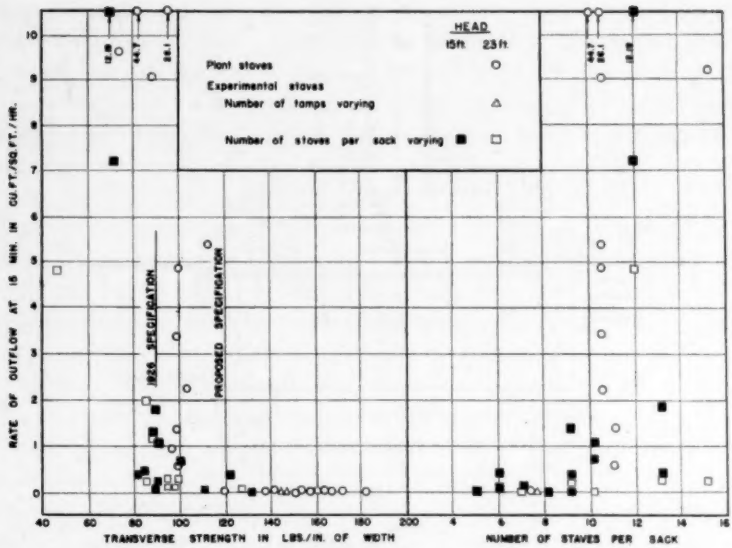

FIG. 2—PERMEABILITY VERSUS THE TRANSVERSE STRENGTH AND THE NUMBER OF STAVES PER SACK OF CEMENT

80 pounds per inch of width the permeabilities of a considerable percentage of the staves are still low.

From the data of the right panel of Fig. 2 low permeabilities appear to be assured if the proportions are such that not more than 8½ staves are made per sack of cement. However, Fig. 2 shows also that it is possible to obtain low permeabilities with much leaner mixes.

Under the low pressures and short periods used in these tests both the aggregates and the cement pastes are impermeable. Consequently, passage of water is through voids resulting from the use of proportions such that the volume of paste (water plus cement) is less than the volume of voids in the aggregate.

Data necessary for the calculation of the air voids not being available, macrophotographs were taken of plane sections perpendicular to the face of the stave. Typical sections are shown in Fig. 3. Examination of these two views will show that (a) when the specimen is of high strength and low permeability, air (or water) voids may exist, but they are isolated, and (b), in specimens of low strength and high permeability, the volume of voids is much greater and the voids are interconnected, thus providing channels for the ready penetration or passage of liquids.

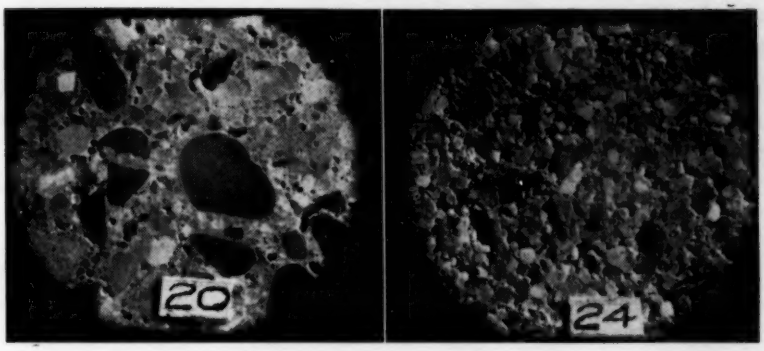

FIG. 3—MACROPHOTOGRAPH OF TWO PLANT STAVES. AS WILL BE READILY SEEN, SPECIMENS 20 AND 24 WERE MADE AT DIFFERENT PLANTS. NOTE THAT THE VOIDS ARE ISOLATED IN SPECIMEN 20 AND CONNECTED IN SPECIMEN 24. MAGNIFICATION ABOUT 2.2

Specimen No. 20
Water-cement Ratio—3.7 gal. per sack
Transverse Strength—152 lb. per inch
Ratio 5 min. to 24 hr. absorption—0.26
Staves per sack—8.1
Permeability—0 cu. ft. per sq. ft. pr hr. at 15 min.
No. of days acid treatment at ⅛ in. depth of scaling—56

Specimen No. 24
Water-cement Ratio— ?
Transverse Strength—83 lb. per inch
Ratio 5 min. to 24 hr. absorption—0.79
Staves per sack—10
Permeability—44.7 cu. ft. per sq. ft. per hr. per 15 min.
No. of days acid treatment at ⅛ in. depth of scaling—36

ACID TESTS

These tests were started in the fall of 1937. For some time the chief object was the development of a suitable test procedure. Later series had for their purpose the evaluation of the relative effect of the numerous variables encountered both in the specimens and in the corrosive solutions. While referred to for convenience as acid tests, some of the solutions used were not acid solutions.

Series A

This series was purely an exploratory investigation for test procedure. The outline included the following variables:

I. Method of Test
   a. Method of Submersion
      (1) Continuous
      (2) Intermittent
   b. Acid
      (1) Hydrochloric
      (2) Acetic
      (3) Lactic
   c. Strength of Acid
      (1) 0.5 Normal
      (2) 0.1 Normal
      (3) 0.05 Normal

II. Mortar Properties

The properties of the four mixes used are given in Table 1.

The above mortars were made from aggregate containing 30 per cent pea gravel and 70 per cent concrete sand. The specimens were 2 in. cubes molded by vibration and cured 7 days in the moist room before subjecting to acid treatment.

TABLE 1—PROPERTIES OF MIXES USED IN SERIES A

Mix	Ratios				Number* of Staves per Sack of Cement	Per Cent Absorption After 24 Hrs. Submersion		Compressive Strength at 7 Days p.s.i.
	Water / Cement, gal. per Sack	Volume of Paste to Volume of Voids in Aggregate	Air Voids per Unit Volume of Concrete	Volume of Concrete to Bulk Volume of Aggregate		By Wt.	By Vol.	
A	3.98	1.89	0	1.25	5.72			12,140
C	4.17	1.124	0.002	1.04	8.30			8,780
D	8.02	0.80	0.045	1.00	16.60	5.71	12.6	4,010
E	8.04	0.60	0.097	1.00	22.15	8.20	16.9	1,285

*Based on computations using the volume of a 2½ x 10 x 30 inch stave as 0.434 cu. ft.

The specimens were submerged in 10 litres of acid solution placed in 5 gal. earthenware crocks. Duplicate specimens were placed at the bottom of the crock and on a copper mesh rack suspenced about 3 in. below the surface of the solution. The specimens on the bottom were left continuously submerged except for removal for weekly weighings. The specimens on the racks were intermittently submerged in the acid solution and left in air using a 24 hour cycle.

At weekly intervals the specimens were removed from the acid solutions, placed in running water for a few minutes, blown off by a standardized air-stream and weighed. The per cent loss in weight based on the original surface dry weight was used as the measure of acid action.

The use of an air stream is believed to be a very satisfactory method for removing deteriorated material. It gives a definite end-point and removes loose material only.

While the larger aggregate particles project from the cubes after submersion in acid solution, the body of the specimen remains as a cube of smaller dimensions but nearly as perfectly formed as when removed from the mold. There is no rounding of the corners as in freezing and thawing tests. The material remaining after blowing off is hard and strong. In other words, the acid penetrates on planes parallel to the surface of specimen.

From titrations made weekly on the fresh and used solutions it was found that the average acid consumption per specimen was about equal when the number of specimens was varied from 6 to 12 per crock. Hence it was concluded that the effect of a limited variation in the

number of specimens per crock was negligible. Because of the possible effect of the salts resulting from the action of the acids on the specimens, it was considered better practice to use fresh solutions each week rather than to renew the solutions by the addition of acid.

The specimens placed on the bottom of the crocks and continuously submerged showed less action than those placed on the racks and intermittently submerged, notably for those specimens immersed in acetic acid. This was at first believed due to the method of submersion but was shown later to be chiefly due to stratification of the acid solutions and the salt solutions. Consequently, the data on the continuously submerged specimens were of no further value and were abandoned.

Relations between the per cents loss in weight and the duration of the acid treatment in days are shown in Fig. 4 for Mixes A and C. The relation for Mixes D and F are similar. It will be seen that, for all practical purposes, the per cent loss in weight is directly proportional to the duration of the acid treatment. Similar relations between acid corrosion and time of treatment were obtained in succeeding series. It will be noted from Fig. 4 also that the difference in resistance of mixes A and C is small despite the considerably greater cement content and strength of mix A.

Using the average per cents loss in weight at 14, 21, and 28 days treatment, ratios of these losses for mixes C, D, and E to those for mix A were calculated. In Fig. 5 these relative values are plotted against the compressive strengths of the various mixes. It will be seen that the weaker specimens are considerably more susceptible to acid attack than the stronger specimens.

After completion of the acid tests a number of specimens were capped and their compressive strengths obtained. The areas used in reducing the compression data were obtained from the per cent losses in weight by a method to be explained later. The ratios of the strengths after acid exposure to the corresponding strengths at the beginning of treatment are given in Table 2.

TABLE 2—RATIOS OF COMPRESSIVE STRENGTHS AFTER ACID TREATMENT TO COMPRESSIVE STRENGTHS AT THE BEGINNING OF ACID TREATMENT

Mix	Acid and Normality					
	Hydrochloric		Lactic		Acetic	
	0.1 N	0.05N	0.1 N	0.05 N	0.1 N	0.05N
A	0.80	0.93	0.87	0.95	0.93	1.07
C	0.91	1.19	0.62	1.11	....	1.29
D	1.24	1.28	1.07	1.27	....	1.05
E	1.49	1.90	....	1.72	....	1.74

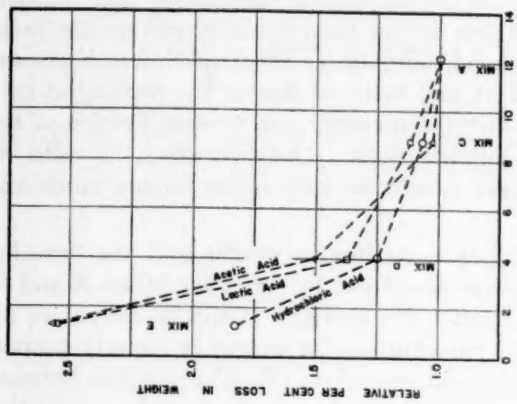

FIG. 4—(LEFT) RELATIONS BETWEEN THE PER CENT LOSS IN WEIGHT AND THE DURATION OF THE ACID TREATMENT IN DAYS

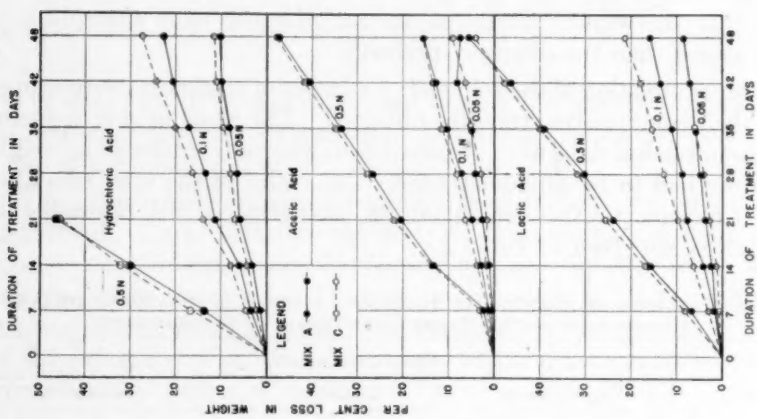

FIG. 5—(ABOVE) THE EFFECT OF THE COMPRESSIVE STRENGTH OF MORTARS ON THE PER CENT LOSS IN WEIGHT DUE TO INTERMITTENT SUBMERSION IN HYDROCHLORIC, LACTIC, AND ACETIC ACID SOLUTIONS

The high strength ratios obtained are in line with the observations made during the tests that the core remaining after blowing off the loosened material appeared hard and strong. No explanation is offered for the higher ratios for the more porous mixes. The reverse was expected at the beginning of the tests. In another series reported elsewhere[1] the strength ratios for dense specimens were less than unity and those for the porous specimens greater than unity. The important inference from Table 5 is that damage due to acid under the exposure conditions of these tests is largely confined to the surface.

Standard Test Procedure for Series B, D, and E

As a result of the experience gained in Series A, the following procedure was adopted for subsequent tests:

Twelve 2-in. cubes or seven 2½ in. cubes were submerged for 24 hours in 0.16N lactic and acetic acid solution on a copper mesh rack suspended about 3 in. below the surface of 12 litres of solution contained in 5 gal. earthenware crocks. Following acid submersion, the cubes were stored in laboratory air for 24 hours. The movement of the specimens in and out of the solutions was so scheduled that the specimens were submerged from Saturday noon until Monday noon. Thus there were three complete cycles per week, one cycle having a 48 hour submersion period.

Prior to acid testing, the specimens were submerged in water for 24 hours, then blown off with a standardized air stream and weighed. This weight was used as the basis for computing subsequent per cent losses in weight. The cubes were blown off and weighed at weekly intervals during the test period.

The 0.16N solution was made by using quantities of lactic and acetic acids such that the normality of each acid was approximately 0.08. This solution was used because data available indicated it to be as close an approach to the juice existing at the end of the fermentation period of corn silage as could be simply obtained and economically used. Titrations made weekly showed the normality of the fresh solution to be about 0.15N. Consequently the 0.16N should be regarded as a nominal value only.

The fact that the shape of the cube remains practically unchanged after acid treatment permits the use of the depth of scaling as a criterion of the relative effect of corrosive solutions. The depth of scaling has been used in field investigations and is a more direct and significant criterion than is the per cent loss in weight. The depth of scaling is easily calculated from the per cent loss in weight. Fig. 6 shows the graph used for this purpose. It will be noted from the graph

[1]For references see end of paper.

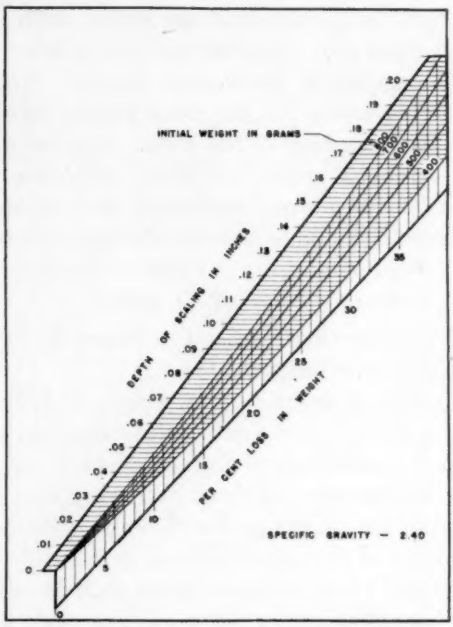

FIG. 6—GRAPH USED FOR CALCULATING THE DEPTH OF SCALING FROM THE PER CENT LOSS IN WEIGHT

that the depth of scaling for a given per cent loss in weight varies with the volume of the specimen. Consequently the depth of scaling gives a truer comparison of the acid attack on specimens of unequal size than does the per cent loss in weight.

The number of days treatment required to cause scaling to a depth of ⅛ in. was taken as the criterion of resistance to corrosive solutions. This depth of scaling corresponds to about 30 per cent loss in weight.

In the acid tests as in the permeability tests all specimens were obtained from staves manufactured by the dry-tamp process. Consequently, conclusions resulting from the data of this paper do not necessarily apply to wet-cast staves.

Series B

The specimens for this series were approximately 2½ in. cubes cut from the part of the half-staves not used in the permeability tests. Two cubes were used per stave.

In Fig. 7 the resistance to acid as expressed by the number of days acid treatment required to cause scaling to a depth of ⅛ in. is compared to the transverse strength, the permeability, and the cement content as expressed by the number of staves per sack.

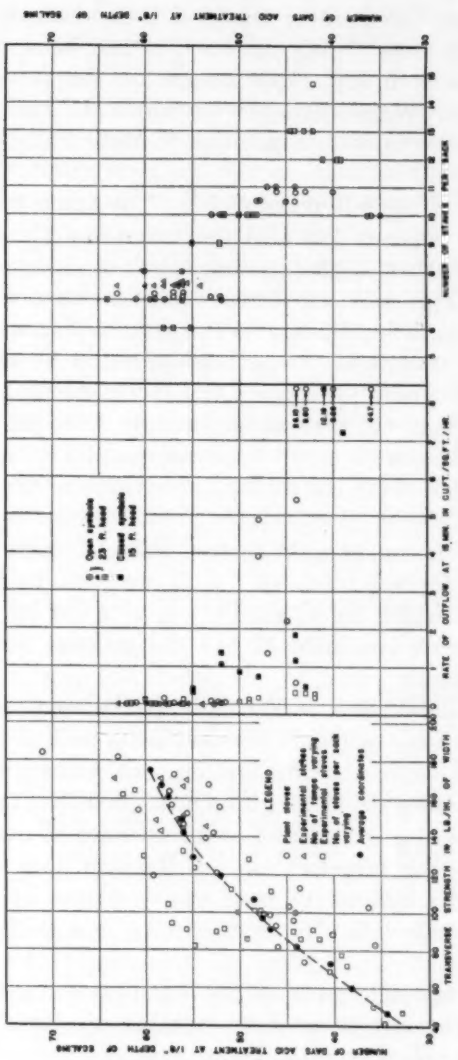
NUMBER OF DAYS ACID TREATMENT AT 1/8" DEPTH OF SCALING
RATE OF OUTFLOW AT 15 MM. IN CU.FT./SQ.FT./HR.
NUMBER OF STAVES PER SACK
TRANSVERSE STRENGTH IN LB./SQ. IN. OF WIDTH
LEGEND
Open symbols 23 ft. head
Closed symbols 15 ft. head
Plant staves
Experimental staves
Experimental staves
Average coordinates

FIG. 7—THE EFFECT OF THE TRANSVERSE STRENGTH, THE PERMEABILITY AND THE NUMBER OF STAVES PER SACK (CEMENT CONTENT) ON THE RESISTANCE TO 0.16N LACTIC AND ACETIC ACID SOLUTION

In examining Fig. 7, it should be kept in mind that the plotted points represent individual tests of transverse strength and permeability and the averages for two cubes cut from the same half-stave for the acid resistance. It is to be expected that whatever relations exist would be much more clearly shown if it had been possible to use the averaged data from, say, 5 like specimens. Because the staves were made at different plants variables of materials, manufacturing methods, and curing are present in addition to those indicated on the figure.

If the dashed line of the left panel of Fig. 7 be taken to represent the average relation between the acid resistance and the transverse strength of the staves, then it appears that (a) the resistance increases with the flexural strength, and that (b) the rate of increase in resistance decreases as the strength increases. So far as acid resistance is concerned no "critical" transverse strength is indicated by these tests. The wide spread of the data for a given transverse strength indicates that variables other than the transverse strength have an important influence on the resistance to acid. Comparison of Fig. 7 with Fig. 5 and 10 will show that these conclusions also apply to the results of Series A and Series E.

The open circle in the upper right corner of the left panel of Fig. 7 represents the data for specimen 72. This was cut from a stave described by Miller[3] which lay at the bottom of a silo for ten years. In spite of this exposure specimen 72 had the greatest resistance to acid of the group tested.

Referring to the middle panel of Fig. 7, it will be seen that though the average acid resistance of highly permeable staves is less than the average resistance of staves having low or zero permeabilities, the range in acid resistance for the latter group is so great that the permeability is of little value as a criterion of acid resistance as measured by the procedure used in this investigation. If the acid tests were made in such a manner that a hydrostatic head was developed by the corrosive solution, better correlation of permeability and acid resistance might be expected.

The data shown in the right panel of Fig. 7 indicates that a relationship exists between the cement content (number of staves per sack) and the acid resistance if other variables are kept constant. However, the range in acid resistance for a given cement content may be so great that the cement content cannot be used alone as a criterion.

Series D

In certain processes of preserving silage phosphoric acid and molasses are used. If improperly handled, the solutions as added may come in

contact with the walls of a silo. Under more careful handling the added solutions will be diluted by the moisture in the silage before coming in contact with the walls. The strengths of the solutions used in this series are believed to represent the strengths of the solutions when added and the estimated strengths after the dilution occuring when they are mixed with the silage.

At the time this series was started a silo filled with phosphoric acid treated "grass" silage was running juice. Enough of this juice was collected for a five week test period. To prevent spoilage, it was stored in a refrigerator until used for test purposes. The solutions and procedures for Series D are given in Table 3.

The purpose of Series D being to compare the effects of the various solutions and modified test procedures with the standard solution and standard procedure (Jar B, Table 3), the loads for each jar were made as nearly alike as possible and consisted of 7 cubes, one from each of 7 staves.

The properties of the staves, the absorptions of 2½ in. cubes and the acid resistance in days of treatment at ⅛ in. depth of scaling for four treatments are given in Table 4. The depths of scaling were less than ⅛ in. for the cubes for the remaining treatments when the series was discontinued at 84 days.

TABLE 3—SOLUTIONS AND PROCEDURES FOR SERIES D

Jar	Solution	Procedure
A	Silage juice, pH 4.95	Submerged 1 day, in air 1 day
B	0.16 N lactic and acetic acid	Submerged 1 day, in air 1 day
C	0.16 N lactic and acetic acid	Continuous submersion
D	0.16 N lactic and acetic acid	Submerged 1 day, in air 6 days
E	0.16 N lactic and acetic acid	Submerged 1 day, in air 1 day but blown off every 3 weeks
F	1 per cent phosphoric acid	Submerged 1 day, in air 1 day
G	1 per cent phosphoric acid	Continuous submersion
H	5 per cent phosphoric acid	Submerged 1 day, in air 1 day
I	800 g. molasses to 12 litres of water	Submerged 1 day, in air 1 day
J	800 g. molasses to 12 litres of water	Continuous submersion
K	2400 g. molasses to 12 litres of water	Submerged 1 day, in air 1 day

It will be noted from Table 4, that the resistances of all cubes to each of the four treatments are nearly equal despite large differences in strength and in the ratio of the 5 minutes to 24 hour absorptions. This result is interpreted as not contradictory to the trends exhibited by the data of Series A, B, and E but is believed to indicate that variables other than the flexural strength and the absorption cannot be neglected. The data do suggest that prolonged curing increases the resistance to acid.

The ratios of the average depths of scaling for all specimens of each jar (except Jar H) to the average for those intermittently submerged

TABLE 4—PROPERTIES OF STAVES; ABSORPTIONS AND ACID RESISTANCE OF 2½ INCH CUBES; SERIES D

Stave No.	Plant	No. of Staves Per Sack of Cement	Age Months	Transverse Strength for 2.50 Inches Thickness Lb. per Inch of Width	Per Cent Absorption by Weight for 2½ Inch Cubes			Number of Days Treatment for ⅛ inch Depth of Scaling			
								Solution and Procedure			
					5 Min. Subm.	24 Hr. Subm.	Ratio 5 Min. / 24 Hr.	0.16N Lactic and Acetic Acid Submerged 1 Day, in Air 1 Day	0.16N Lactic and Acetic Acid Continuous Submersion	0.16N Lactic and Acetic Acid Submerged 1 Day, in Air 1 Day, Blown Every 3 Weeks	5% Phosphoric Acid Submerged 1 Day, in Air 1 Day
73	A	9½	14	119	2.4	3.9	0.62	66	77	84	27
74	A	10¾	14	107	2.8	4.2	0.65	66	82	72	26
75				115	2.4	3.9	0.62	67	80	81	28
Average				110	2.6	4.0	0.63	66	81	76	27
76	B	9	2	108	2.9	4.7	0.62	66	75	78	28
77	C	7½	1	166	1.3	4.2	0.31	65	75	78	24
78				152	1.3	4.3	0.30	65	74	78	25
79				161	1.1	4.3	0.26	63	74	74	25
Average				162	1.2	4.3	0.29	64	74	77	25

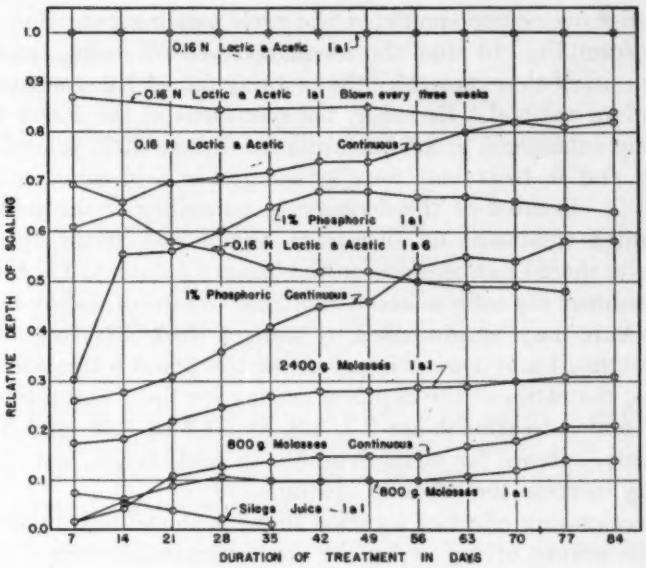

FIG. 8—RELATIVE DEPTHS OF SCALING OF 2½ IN. CUBES CUT FROM SILO STAVES WHEN TREATED WITH THE SOLUTIONS AND PROCEDURES INDICATED ON THE FIGURE

"1"and"1" indicates that the specimens were submerged in acid solution 1 day and were in air 1 day. "1 and 6" indicates that the specimens were submerged in acid solution 1 day and were in air 6 days. "Continuous" indicates that the specimens were continuously submerged. All specimens were blown off and weighed every week except the one set marked "blown every three weeks." The pH of the silage juice was 4.95. The plotted points are the averages for 7 cubes.

in 0.16N lactic and acetic acid are plotted in Fig. 8. It will be seen that these ratios for some solutions, notably for the phosphoric acid solutions, vary considerably with the duration of the treatment but become more or less constant from the 70th to the 84th day of treatment. Averages for these periods have been used in comparing the corrosive effect of the various solutions (see Table 7).

The procedure for Jar E, i.e., blown off every three weeks instead of every week, was adopted in order to find out if corroded material gave protection to the underlying mortar. The graph for the specimens blown every three weeks indicates that acid corrosion is retarded by the scale. Presumably greater retardation would occur if the periods between blowing were increased.

In previous series it has been observed that the depth of scaling increased when specimens were left in air following the completion of the acid tests due to the loosening of surface material. This action suggested the use of the 1 day submerged 6 days in air treatment. It was thought also that a cycle having a short acid period and a long

air period more nearly simulated the cycle existing in a silo. It will be seen from Fig. 10 that the average depth of scaling was about fifty per cent of that caused by the in 1 day out of 1 day treatment of the standard method. However, the specimens of the 1 and 1 treatment were submerged in acid solution for 4 days in 7, whereas those of the 1 and 6 treatment were submerged in acid solution for but 1 day in 7. Because of the decrease in normality of the solution of the 1 and 1 treatment on successive immersions of the specimens, comparison should not be made on the basis of 4 days to 1. Assuming a 3 to 1 submersion ratio as about right, the depths of scaling for equal acid exposure may be compared by using 1 week intervals for the 1 and 1 treatment and 3 week intervals for the 1 and 6 treatment. On this basis, the ratios of the depth of scaling for the 1 and 6 treatment to the 1 and 1 treatment are 2.2, 1.8, and 1.7 at 3, 6, and 9 weeks respectively. Hence for equal exposure to acid, longer periods in air materially increase the depth of scaling.

The accelerating effect of a period in air is shown also by comparison of the graphs of Fig. 8 for the continuously submerged and for the 1 and 1 treatments using 0.16N lactic and acetic acid and 1 per cent phosphoric acid. The 800 g. molasses solution was an exception in that the corrosive action was greater than the specimens were continuously submerged, which may be explained by the fact that the specimens of this treatment did not become surface dry during the 24 hours in air.

It will be seen from Fig. 8 that the silage juice had little effect on the specimens. Because this juice was running from a silo in which phosphoric acid had been used as a preservative it was expected to be strongly acid. However, its pH value was found to be 4.95 which is much higher than that of juice within well preserved silage. Apparently the corrosive action of silage juices having pH values of 4.95 or greater can be neglected when considering the exposure conditions in a silo.

Series E

The object of Series E was to compare the corrosive action of the 0.16N lactic and acetic acid solution with those of corn juice and finished silage juice.

The corn juice and finished silage juice were both expressed from the material stored in the same silo, the corn juice being obtained as the silo was filled and the silage juice as the silage was fed. All corn juice was obtained necessarily during the filling period and then was stored in a refrigerator until needed for test purposes. The silage juice was expressed when required.

Because bacterial action occurs when either corn juice or silage juice at room temperature is exposed to air, the space in the jars above the liquid level was kept full of carbon dioxide gas.

TABLE 5—SOLUTIONS AND PROCEDURES FOR SERIES E

Jar	Solution	Procedure	Solution Changed at
A	0.16 N lactic and acetic acid	Submerged 1 day, in air 1	7 days
B	0.16 N lactic and acetic acid	Continuously submerged	7 days
C	Corn juice, pH 4.3	Submerged 1 day, in air 1	14 days
D	Corn juice, pH 4.3	Continuously submerged	14 days
E	Silage juice, pH 3.7	Submerged 1 day, in air 1	7 days
F	Silage juice, pH 3.7	Continuously submerged	7 days
G	Silage juice, pH 3.7	Submerged 1 day, in air 1	14 days
H	Silage juice, pH 3.7	Continuously submerged	14 days

The solutions and methods of procedure for Series E are given in Table 5. The 14 day period for changing solutions was selected in order to permit time for completion of the normal reactions occurring within corn juice at room temperature.

The specimens were 2½ in. cubes cut from half-staves about three months old at the beginning of testing. They had been made at the same plant and were not subject to unmeasured variables of curing, materials, etc. which had been present in all other groups of staves tested.

As in Series D the loads for each jar consisted of one cube from each of 7 half-staves. The properties of the staves and the data on cubes cut therefrom are given in Table 6.

Using the average depth of scaling for the 7 cubes of each jar, the relative depths of scaling were calculated and plotted in Fig. 9. The graphs for the silage juice are approximately parallel to those for the 0.16N lactic and acetic acid. Those for the corn juice are more or less parallel up to 35 days treatment and then slope downward to meet the graphs for the silage juice. At the time the tests were being run it was noted that the color of the corn juice changed and that the pH value decreased to that of the silage juice. Evidently the temperature of the storage refrigerator had not been low enough to prevent the fermentation of the corn juice so that during the last half of the test period it had turned into silage juice.

It will be noted from Fig. 9 that the corn juice is approximately 50 per cent more corrosive than the finished silage juice. Because the period of most abundant leakage is during filling, it is this solution which causes most of the damage on the exterior of a leaking silo.

The effect of the CO_2 used to obtain an anerobic condition in the jars containing corn and silage juices was not determined in this

TABLE 6—THE PROPERTIES OF STAVES USED IN SERIES E

Stave No.	No. of Staves per Sack of Cement	Consistency	Transverse Strength for 2.5 In. Thickness. Age 2 Mo. Lb./In. of Width	Per Cent Absorption, by Weight on 2½ Inch Cubes					Relative Depths of Scaling at 56 Days (Average 49 to 63 Days)												Average Corn Juice and Silage Treatments
									0.16N Lactic and Acetic Acid			Corn Juice*			Silage Juice						
									Sol. Changed at 7 Days			Sol. Changed at 14 Days			Sol. Changed at 7 Days			Sol. Changed at 14 Days			
				5 Min. Subm.	10 Min. Subm.	24 Hr. Subm.	72 Hr. Subm.	Ratio 5 Min. / 24 Hr.	Subm. 1 Day, in Air 1 Day	Continuously Subm.	Average	Subm. 1 Day, in Air 1 Day	Continuously Subm.	Average	Subm. 1 Day, in Air 1 Day	Continuously Subm.	Average	Subm. 1 Day, in Air 1 Day	Continuously Subm.	Average		
87	11	Normal	105	4.6	4.8	5.5	5.6	0.84	1.40	1.32	1.36	1.35	1.95	1.65	1.50	1.95	1.72	1.20	1.80	1.50	1.62
85	8½	Normal	146	2.2	2.5	4.6	4.7	0.48	1.13	1.13	1.13	1.10	1.20	1.15	1.22	1.14	1.18	0.95	1.41	1.18	1.17
86			140	3.1	3.5	4.9	5.0	0.63	1.33	1.12	1.22	0.97	1.56	1.26	1.20	1.42	1.31	1.10	1.43	1.26	1.27
Aver.			143	2.7	3.0	4.7	4.8	0.55	1.23	1.12	1.17	1.03	1.38	1.20	1.21	1.28	1.24	1.02	1.42	1.22	1.22
82	7	Normal	167	1.1	1.4	4.2	4.3	0.25	1.12	1.0	1.06	0.93	1.25	1.09	1.37	1.0	1.18	0.92	1.00	0.96	1.08
88			165	1.0	1.3	4.3	4.5	0.23	1.21	1.19	1.20	1.07	1.45	1.26	1.32	1.25	1.28	1.08	1.26	1.17	1.24
Aver.			166	1.0	1.3	4.2	4.4	0.24	1.16	1.09	1.13	1.00	1.35	1.17	1.34	1.12	1.23	1.0	1.13	1.06	1.16
83	7	Dry	108	3.6	3.9	5.4	5.6	0.67	1.20	1.27	1.23	1.04	1.55	1.29	1.27	1.35	1.31	0.90	1.26	1.08	1.23
84	7	Wet	224	1.0	1.4	4.7	3.8	0.21	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

*Relative depths of scaling at 28 days (average 21 to 35).

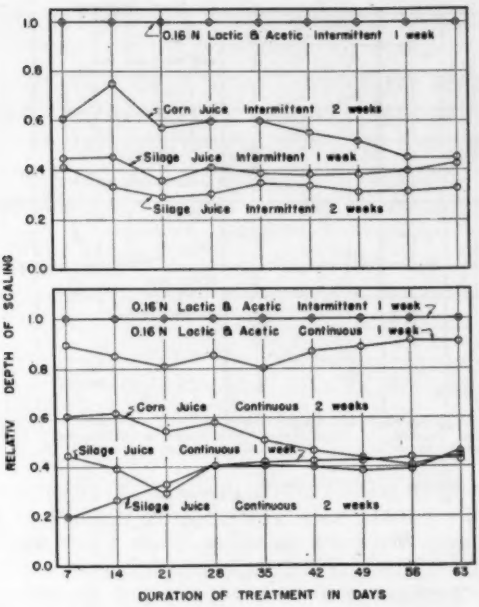

FIG. 9—DEPTHS OF SCALING FOR 2½ IN. CUBES CUT FROM STAVES AND TREATED WITH CORN JUICE AND SILAGE JUICE RELATIVE TO THE DEPTHS OF SCALING FOR SIMILAR CUBES TREATED WITH 0.16N LACTIC AND ACETIC ACID SOLUTION

The pH average for the corn juice was 4.3 and that for the silage juice 3.7. It is thought that the corn juice only existed as such up to about 35 days, after which time it gradually changed to silage juice. Each plotted point represents the average for 7 cubes.

investigation. Because CO_2 is present in large quantities during and after filling the silo, its use is thought to make the exposure condition of the tests more nearly similar to that occurring in a silo.

Relations between the relative depths of scaling and the transverse strengths, consistencies, numbers of staves per sack of cement, and the ratios of the 5 minute to the 24 hour absorptions are shown on the upper part of Fig. 10. The points plotted represent the averages for the specimens treated with 0.16N lactic and acetic acid and with corn and silage juice. The similarity of the two graphs of each panel of the upper half of Fig. 10 and the parallelism of the graphs of Fig. 9 are interpreted as proof that a lactic and acetic acid solution may be used to determine qualitatively the relative resistance of mortars to silage juices. It will be noted that the spread in the relative depths of scaling due to differences in the quality of the mortars is greater for the silage juices than for the lactic and acetic acid solutions. This

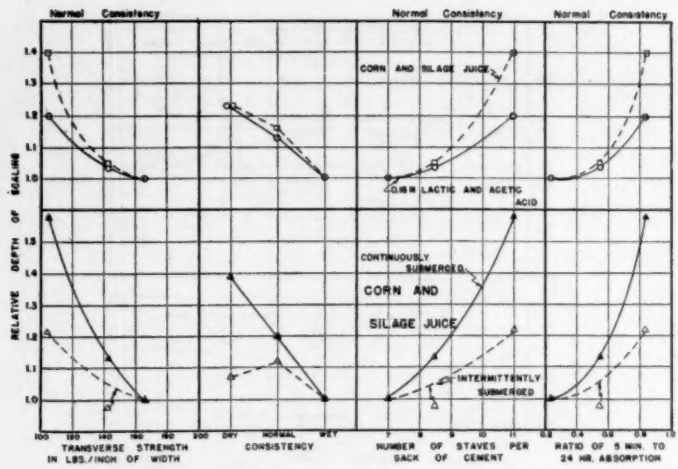

FIG. 10—THE UPPER HALF OF THE DIAGRAM COMPARES THE CORROSIVE ACTION OF 0.16N LACTIC AND ACETIC ACID SOLUTION WITH THAT OF CORN AND SILAGE JUICE ON SPECIMENS OF VARYING QUALITY. THE POINTS FOR THE 0.16N LACTIC AND ACETIC ACID SOLUTION ARE THE AVERAGES FOR 2 CUBES AND THE POINTS FOR THE CORN AND SILAGE JUICE ARE THE AVERAGES FOR 6 CUBES. THE LOWER HALF OF THE FIGURE COMPARES THE RELATIVE RESISTANCE OF SPECIMENS OF DIFFERENT QUALITY WHEN (A) INTERMITTENTLY SUBMERGED IN SILAGE JUICES AND (B) CONTINUOUSLY SUBMERGED IN SILAGE JUICES

fact should be kept in mind when attempting to apply the data of tests using this solution to the relative life of staves in service.

The relative depths of scaling used in plotting the graphs of the lower half of Fig. 10 are the averages for the cubes continuously submerged and intermittently submerged in corn juice and in silage juice. These graphs still further emphasize the need for caution in the interpretation of acid tests since they show very clearly that the relative resistance of staves of different quality is affected materially by the exposure conditions.

Examination of Table 6 will show that there is considerable variation between the data for individual specimens subjected to the same treatment. This variation is believed to be due in part at least to variations in the quality of individual cubes. Similar variations have been noted when making absorption tests. Consequently it is evident that good relationships between the transverse strength and other properties of the stave will be obtained only if a sufficient number of cubes are used to properly sample the stave.

The evidence of Fig. 10 is similar to that of Fig. 5 and 7 in that the

resistance to corrosion increases as the strength and cement content increase.

The various solutions used in Series D and E are arranged in Table 10 in the order of their corrosive attack on specimens cut from dry-tamped staves. The relative depths of scaling were those obtained by using the average depth of scaling for 70, 77 and 84 days of treatment for the specimens of Series D and for 21, 28 and 35 days treatment for Series E.

TABLE 7—RELATIVE CORROSIVE ACTION OF THE SOLUTIONS USED IN SERIES D AND E

Solution	Procedure	Relative Depths of Scaling
5% phosphoric acid	In 1 day, out 1 day	3
0.16 N lactic and acetic acid	In 1 day, out 1 day	1
0.16 N lactic and acetic acid	Continuous submersion, Series D	0.8
0.16 N lactic and acetic acid	Continuous submersion, Series E	0.8
0.16 N lactic and acetic acid	In 1 day, out 1 day but blown every 3 weeks	0.8
1% phosphoric acid	In 1 day, out 1 day	0.6
Corn juice pH 4.3	In 1 day, out 1 day, solution changed at 14 days	0.6
1% phosphoric acid	Continuous submersion	0.6
Corn juice, pH 4.3	Continuous submersion	0.5
0.16 N lactic and acetic acid	In 1 day, out 6 days	0.5
Silage juice, pH 3.7	In 1 day, out 1 day, solution changed at 7 days	0.4
Silage juice, pH 3.7	Continuous submersion, solution changed at 14 days	0.4
Silage juice, pH 3.7	Continuous submersion, solution changed at 7 days	0.4
Silage juice, pH 3.7	In 1 day, out 1 day, solution changed at 14 days	0.3
2400 q. molasses	In 1 day, out 1 day	0.3
800 q. molasses	Continuous submersion	0.2
800 q. molasses	In 1 day, out 1 day	0.1
Silage juice, pH 4.9	Continuous submersion	0.02

The ratios have been rounded to one significant figure in order to avoid any misinterpretation of their accuracy. At best they are a rough index of the relative corrosive effect of the various solutions.

Absorption Tests

The significance of the 24 hour absorptions of mortars and concrete has long been questioned. [2] The use of the absorption after ten minutes submersion has been suggested by Miller [3] and the ratio of the absorption at ten minutes to the absorption at 24 hours by Wendt and Woodworth. [4]

In order to examine the usefulness of these three absorption values as criteria of the permeability, strength, and acid resistance of mortars, the per cent absorptions by weight of 2½ in. cubes cut from the staves used in the permeability tests and the acid tests of Series B were obtained for 5 minute and 24 hour submersion following oven-drying at 215 to 220° F. These data, and the ratios of the 5 minute to 24 hour absorptions are plotted in Fig. 11 against the permeability, the acid resistance, and the transverse strength.

Examination of Fig. 11 will show that the ranges of values for both the 5 minute absorption and the ratio of the 5 minute absorption to

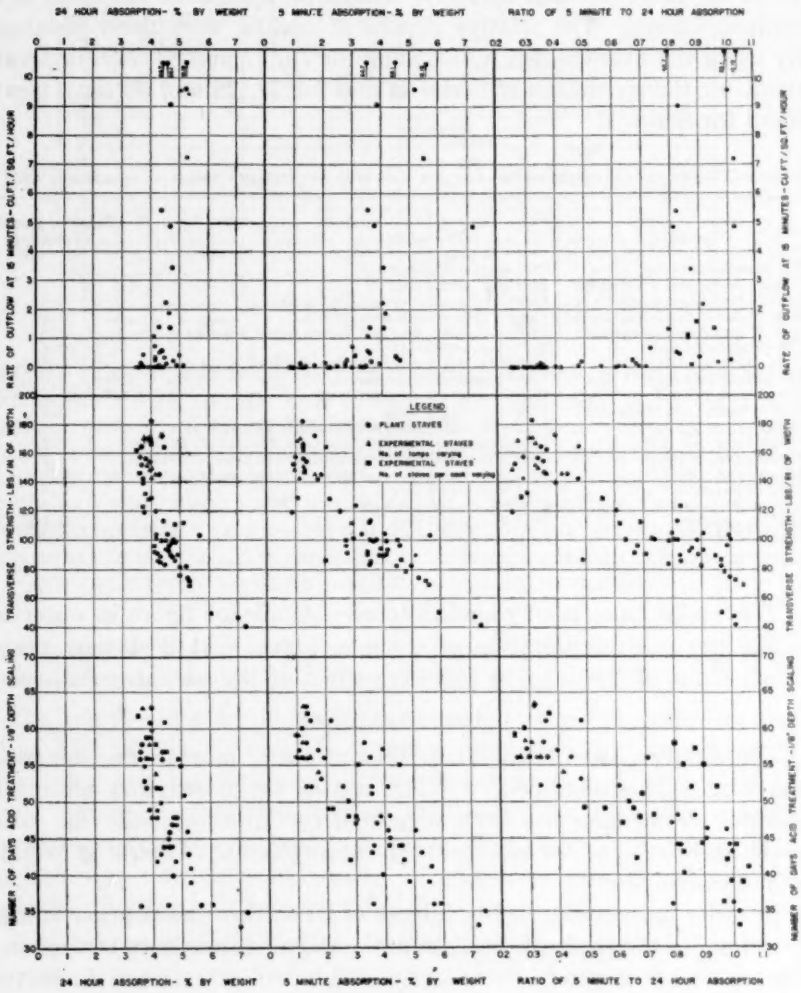
24 HOUR ABSORPTION - % BY WEIGHT
5 MINUTE ABSORPTION - % BY WEIGHT
RATIO OF 5 MINUTE TO 24 HOUR ABSORPTION
RATE OF OUTFLOW AT 6 MINUTES - CU.FT./SQ.FT./HOUR
TRANSVERSE STRENGTH - LBS./IN OF WIDTH
NUMBER OF DAYS ACID TREATMENT - 1/8" DEPTH SCALING
LEGEND
PLANT STAVES
EXPERIMENTAL STAVES
No. of lumps varying
EXPERIMENTAL STAVES
No. of staves per test varying

FIG. 11—RELATIONS BETWEEN THE PERMEABILITY, TRANSVERSE STRENGTH, AND ACID RESISTANCE AND THREE ABSORPTION CRITERIA

the 24 hour absorption are much greater than that for the 24 hour absorption and, therefore, are better criteria.

For the specimens of this investigation, a limit of 0.65 for the ratio of 5 minute to 24 hour absorption would exclude all staves of high permeability. With one exception no stave so excluded had a transverse strength greater than 120 lb. per inch of width. The permeability for this stave was 0.42 cu. ft. per sq. ft. per hour and the transverse strength 123 lb. per in. of width. Except for a group of specimens cut from experimental staves, all specimens having a value for this ratio in excess of 0.65 were also low in resistance to acid. When all three relations for each absorption criterion are considered, it is believed that the ratio of the five minute to the 24 hour absorption is the most useful though its superiority over the five minute absorption is not marked.

HYDROSTATIC PRESSURES IN SILOS

If the volume and location of juice in a silo combine to develop hydrostatic pressure at the wall, not only may there be an abundant volume of juice available for reaction with the silo but in all probability, the movements of the juice are also such as to bring fresh juice continually to the wall. Furthermore, the penetration of juices into the wall will increase with increase of hydrostatic head. Data on hydrostatic pressures in silos should, therefore, be helpful in any attempt to evaluate the conditions of exposure of silo walls to silage juices.

In the fall of 1938 during the filling, four open tubes were inserted through the wall of a wood silo about 3 feet above the base and equally spaced around it. No liquid appeared in these tubes.

Another effort to demonstrate and to measure hydrostatic heads in silos was made in the summer of 1939 at a cast in place concrete silo which was being filled with reed canary grass, alfalfa and sweet clover treated with 7 gallons of phosphoric acid, 20 gallons of molasses and 12 gallons of water for each 6 tons of silage. Because the season was wet and high hydrostatic heads could be expected, manometers filled with mercury were used instead of open tubes. The manometers were attached to steel pipes driven into holes drilled through three steel doors of the silo at 5 ft.-8 in., 9 ft.-5 in. and 13 ft.-6 in. above the floor of the silo. The two lower manometers indicated zero pressure differences throughout the period of observation. Seven days after filling, the upper manometer showed a negative pressure of 2.7 in. of mercury (36.7 in. of water). The manometer tube was removed from the pipe and a wire poked through into the silage to make sure there was no obstruction in the pipe. When the manometer was reattached the initial reading checked. At 15 days, the manometer again indi-

cated a negative pressure of 1.70 in. of mercury (23 in. of water). No leakage occurred from this silo.

TABLE 8—MANOMETER PRESSURES IN INCHES OF WATER MEASURED ON A CONCRETE STAVE SILO FILLED WITH GREEN CORN

Days from Start of Filling	Height of Leakage above Base Ft.	Height of Fill above Base Ft.	Manometer No. 1 6′-4″ above Base	Manometer No. 2 9′-2″ above Base	Manometer No. 3 11′-8″ above Base	Manometer No. 4 14′-2″ above Base
3*	21	35	0	0	0	0
3.1	23½	41	+1.6	+3.4	0	−1.4
4	23½	36½	−2.7	−2.5	−20.4	−2.6
4.8	13½	36	−3.3	−5.7	−19.5	−2.6
5	21	42	−5.4	−3.0	Reset***	−1.4
5.2**	33½	47	−5.4	−5.2	+1.2	+0.3
6	36	43	−5.4	−4.2	+1.0	0
7	33½	42	−4.1	−2.4	−1.8	+0.3
8	21	41½	−4.1	−3.1	−5.7	+1.4
9	21	41	0	−8.7	+2.3	+0.3
10	18½	40½	0	−6.5	+1.2	−0.8
11	16	40	0	−6.5	−4.5	−1.6
12	16	39¾	−2.7	−3.8	−3.5	−3.5
13	16	....	−4.1	−5.2	+5.7	−4.9
14	....	....	−4.4	−6.5	−0.4	−5.2
22	....	....	−5.7	−51.2***	−0.7	−3.5
29	....	....	−4.3	−47.5	+0.9	−5.2
34	....	....	....	−6.8	−1.8	−5.2
52	....	36	....	+1.4	−1.8	....

*Manometers placed on silo.
**Silo filled.
***Manometer tube removed to check on initial reading of manometer.

In July, 1939, six manometers were attached to tubes inserted into the wood silo previously referred to. Four of these manometers were located about 3 feet above the concrete base and the other two about 12 feet above the base. The silo was filled with green alfalfa and topped with green sweet clover. No hydrostatic pressures were indicated by any manometer during or after filling. Slow leakage which started from cracks in the base 5 days after the filling began continued for at least one month.

Four manometers were placed on the doors of a concrete stave silo filled with green corn in September, 1939. Leakage from this silo was so great that it was truly a "boiler" to use a term common in the silo industry. The leakage came from around the doors and from four vertical cracks in the walls. The maximum height of leakage through the cracks was about five feet from the top of the silo.

The vertical cracks in the silo walls were easily evident in the empty silo particularly on the inside, which had a thin coat of cement mortar on the lower half and a cement wash over the full height.

The corrosion at these cracks where the silage juice bubbled out and in the path of the flowing juice was so rapid and noticeable that the silo manufacturer coated the outside of the silo with a cement wash about six weeks after the silo was filled.

The hydrostatic pressures obtained on the four manometers are

given in Table 8, from which it will be seen (a) that at any given point the pressures vary widely and erratically, (b) negative pressures are more common than positive pressures and at times are high. The highest negative pressure, 51.2 in. of water corresponds to 266 lb. per sq. ft. which equals the pressure resulting from 24.2 ft. of silage when calculated by use of the common assumption that the pressure due to corn silage is equal to that of a liquid weighing 11 lb. per cu. ft.

The resetting of the manometers indicated in Table 8 and mentioned previously was done to demonstrate that the negative pressures indicated by the manometers really did exist.

The data presented are too few to be valuable for more than to indicate the need of investigations of the exposure conditions existing in silos.

CONCLUSIONS

Though it is recognized that the number of specimens relative to the number of variables was small, it is believed that the data presented lead to and, in many instances, justify the following conclusions for *dry-tamped* concrete silo staves:

1. High permeabilities under low heads are found when the proportions are such that the ratio of the volume of paste (water plus cement) to the volume of voids in the aggregate is low.
2. For exposure conditions similar to those used in these tests, corrosion due to acid solutions is largely confined to the surface.
3. Other variables being constant, the resistance of mortars to acid corrosion increases with increase in transverse strength. Though the rate of increase in resistance decreases with increase in transverse strength no "critical" strength was apparent from the data of this investigation.
4. Other variables being constant, the resistance to corrosion increases with increase in the cement content.
5. There is little, if any, correlation between permeability and the resistance of mortars to acid solutions when no hydrostatic head exists.
6. Corroded surface material provides some protection to the underlying concrete.
7. For equal exposures to acid solutions, the depth of sealing is increased with increase in the length of the air exposure period between submersions.
8. The corrosive action of silage juice having a *pH* value of about 3.7 is approximately 40 per cent of that of 0.16N lactic and acetic acid solution. The corrosive action of silage juice having a *pH* value of 4.95 or higher is negligible.

9. The corrosive action of corn juice is greater than that of corn silage juice.

10. Corrosion resulting from immersion in a weak lactic and acetic acid solution is qualitatively similar to that resulting from immersion in corn or corn silage juice.

11. Quantitatively the relative resistances of mortars of different quality vary considerably with the exposure condition and with the corrosive solution.

12. The 24 hour absorption is of little value as a criterion of quality.

13. The ratio of the 5 minute absorption to the 24 hour absorption is sensitive to variations in permeability, transverse strength, and acid resistance, and therefore is a useful criterion of mortar quality though not markedly superior to the 5 minute absorption.

14. Further investigation of the exposure conditions existing at the walls of silos is needed before attempting to interpret data from laboratory tests in terms of the relative life of staves in the silo.

ACKNOWLEDGEMENTS

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Discussion, to close in the September Supplement, should reach A. C. I. Secretary by July 20.

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Effect of Impact on Reinforced Concrete Beams*

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MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

The impact resistance of reinforcing steels of various grades was investigated by means of tests on 10″ x 16″ beams on a span of 8 ft., reinforced with varying amounts of steels of different grades. These tests indicate the great value of even small amounts of reinforcement in beams subjected to impact, the enormous reserve against collapse after the yield point of the reinforcement has been passed, and the fact that, within the limits of the tests, when subjected to energy loads even more severe than are likely to be met with in practice no noticeable difference in strength was shown by beams reinforced with different grades of steel.

INTRODUCTION

The use of high yield point steel, particularly rail steel, has often been objected to on the grounds that it is brittle and, therefore, unfit for service as the reinforcement of members which may be subjet to impact. Yet many instances in which concrete buildings have been damaged by accident,‡ or where they have deliberately been torn down, definitely point in the opposite direction. The tests conducted at the University of Delaware, and reported in this paper, were undertaken with the purpose of determining, if possible, something definite regarding the behavior, under impact, of reinforcing steels of various grades.

It has been suggested that the Izod tension impact test should be a reliable guide to the shock resistance of any given grade of reinforcement. It has also been suggested that the area under the tension

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‡Western Newspaper Union, A. & P. Warehouse (Pittsburgh), Ohio State Office Building, etc.

stress-strain curve could well serve a similar purpose. To ascertain the reliability of either of these methods as a criterion for the shock resistance of reinforcing steel, such a double preliminary investigation was first carried out on steels of structural, rail, and extra-high yield point grades, and the results obtained form the first part of this report.

The second part of the report is given over to an analytical study of impact on reinforced concrete beams, in which the impact is considered to be inelastic, that is, after contact, beam and weight move together. The formulas developed by Homersham Cox* for homogeneous beams are adapted to reinforced concrete by means of Maney's equation.** In this form they will be more easily recognized by American engineers than in the more elaborate discussions in which the elastic up and down motion of beam and weight are taken into account.

The third and last part of the report includes the study of the behavior of reinforced concrete beams under the drop hammer. Here comparisons are made between the blow required to produce a certain stress within the elastic range, as computed by the formulas of Part II, and the blow required to cause failure; and the relative merits, as regards resistance to shock, of reinforcing steels of different grades are pointed out.

PART I—IZOD TESTS

A detailed discussion of the results of the Izod tests is given in the Proc. Highway Research Board, 18th Ann. Meeting, 1938, Chap. X, and need not be repeated here. The more significant points are summarized below.

1. When a tension specimen is just broken in one blow in an Izod machine, less total energy is required than when repeated blows of less energy each are used.

2. The energy required to cause rupture in one blow increases with increase in the yield point, but at a less rapid rate.

3. The lightest blow required to cause measurable permanent set is about one-twentieth of that required to break the specimen in one blow.

4. With the extra-high yield point steel, the total elongation, whether under impact or under the slow motion of the usual tension-

*Todhunter and Pearson, p. 771.
**See paper by G. A. Maney, 17th annual meeting of the A. S. T. M.

testing machine, is nearly constant, whereas structural steel, and to a lesser degree rail steel, are deformed much more under repeated blows of low energy than in the usual tension test. The significance of this fact will become apparent in the discussion of the beam tests.

5. Except for (4) above, as yet there seems to be no connection between the strength of a given steel in an Izod test and its strength as reinforcement.

Part II—Analytical Treatment

The following expression* gives the deflection of a homogeneous beam when the mass of the beam itself is disregarded,

$$\Delta = \Delta_1 + \Delta_1 \sqrt{1 + \frac{2h}{\Delta_1}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which Δ is the maximum deflection of the beam due to impact, Δ_1 is the deflection caused by the weight, W, applied as a static load, and h is the height of fall. From this, on the assumption that $\frac{S}{S_1} = \frac{\Delta}{\Delta_1}$, the equation for stress in a beam due to an energy load when the mass of the beam is disregarded, is

$$S = S_1 + S_1 \sqrt{1 + \frac{2h}{\Delta_1}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Here S is the stress under impact and S_1 is the stress produced by the weight of the hammer as a static load.

When the mass of the beam is not disregarded, equation (3) is an expression for its deflection:

$$\Delta = \Delta_1 + \Delta_1 \sqrt{1 + \frac{2h}{\Delta_1} \frac{1}{1 + \frac{17W_1}{35W}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in which W_1 is the weight of the beam and W the weight of the falling hammer.† Stresses due to this deflection may be found from equation (4A).

$$S = S_1 + S_1 \sqrt{1 + \frac{2h}{\Delta_1} \frac{1}{1 + \frac{17W_1}{35W}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (4A)$$

Pictorially Eq. (4A) may be studied as follows. For a beam of constant cross section and with a given falling weight, S_1, Δ_1, and

*Seeley, Resistance of Materials, 2nd Ed., p. 286.
†After Timoshenko, Strength of Materials, part I, p. 313, using symbols employed by Seeley.

$\frac{1}{1+\frac{17W_1}{35W}}$ are constants. If the latter be represented by m, we have, by transposition

$$S - S_1 = S_1\sqrt{1+\frac{2h}{\Delta_1}m} \quad \ldots\ldots(5)$$

Squaring

$$(S - S_1)^2 = S_1^2\left(1+\frac{2h}{\Delta_1}m\right) = \frac{2mS_1^2}{\Delta_1}\left(h+\frac{\Delta_1}{2m}\right) \quad \ldots\ldots(6)$$

which is in the standard form of the parabola

$$(S - S_1)^2 = c(y - k) \quad \ldots\ldots(7)$$

in which

$$c = \frac{2mS_1^2}{\Delta_1}$$

and

$$k = \frac{-\Delta_1}{2m}$$

The graph of equation (6) is shown in Fig. 1, which gives the relation between the height of fall and the corresponding unit stress in the extreme fiber of the flexed member. This curve remains below the horizontal axis from $S = O$ to $S = 2S_1$, because the stress caused by a "suddenly applied load," which may be described as an impact load in which $h = O$, is $2S_1$.

Now the usual equations for stresses in reinforced concrete members are based on the same assumptions as to the linear variation in flexural stress and neglect of distortions caused by shear as are those in members of a homogeneous material. Moreover, at stresses within the elastic range, the elastic curve of a reinforced concrete beam will resemble fairly closely that of a homogeneous beam. Fig. 1, therefore, may be regarded as showing the relation between unit stress in the reinforcement and height of fall.

In predicting deflections from equation (3) or stresses from equation (4A), it remains only to find a value for Δ_1. In a reinforced concrete beam this may be done quite satisfactorily by means of Maney's equation*

$$\Delta_1 = c\frac{L^2}{d}(e_c + e_s) \quad \ldots\ldots(8)$$

*See paper by G. A. Maney, 17th annual meeting of the A. S. T. M.

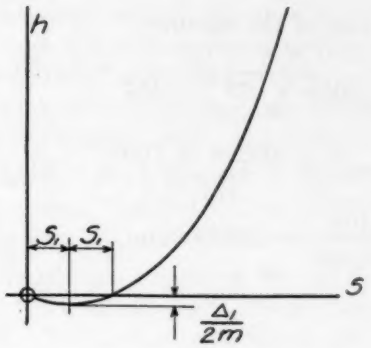

FIG. 1

in which

e_c = unit deformation in extreme fiber of concrete $= \frac{f_c}{E_c}$.

e_s = unit deformation in steel $= \frac{f_s}{E_s}$.

c = a constant depending upon the type of loading, in this case equal to 0.0833.

Equation (4A), which may be written

$$S = S_1 + S_1 \sqrt{1 + \frac{2hm}{\Delta_1}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4B)$$

is the basic equation for making certain comparisons in the tables which follow. As a first example of its use, let it be required to find the distance from which a 2040 lb. hammer must fall onto a 10 x 16 simple beam of 8 ft. span, reinforced with one $\frac{5}{8}$-in. round reinforcing bar whose center is $1\frac{1}{4}$ in. from the lower surface, in order to produce f_s = 45000 p.s.i. The concrete is to be of such quality that $n = 7\frac{1}{2}$ and E_c = 4,000,000 p.s.i.

In this beam $p = \frac{.3068}{10 \text{ x } 14\frac{3}{4}} = .00208$

Substituting in the equation

$$k = \sqrt{(pn)^2 + 2\,pn} - pn \text{ gives } k = .157$$

Under the static load of the hammer

$$f_s = \frac{2040 \times 8 \times 12}{4 \times .3068 \times .95 \times 14\frac{3}{4}} = 11400 \text{ p.s.i.}$$

and

$$f_c = \frac{2pf_s}{k} = \frac{2 \times .00208 \times 11400}{.157} = 302 \text{ p.s.i.}$$

$$e_s = \frac{11400}{30{,}000{,}000} = .00038 \quad \text{and} \quad e_c = \frac{302}{4{,}000{,}000} = .0000755$$

$$e_s + e_c = .00046$$

From these

$$\Delta_1 = .0833 \frac{96^2}{14\frac{3}{4}} (.00046) = .0239'' \quad \text{(see Eq. 8)}$$

$$\text{now } m = \frac{1}{1 + \frac{17}{35} \times \frac{1280}{2040}} = .767$$

Substituting in Eq. 4B gives

$$45000 = 11400 + 11400 \sqrt{1 + \frac{2h \times .767}{1.0239}}$$

from which

$$h = .12''$$

It is thus seen how the effects of any impact load may be converted into steel or concrete fiber stresses. To be sure the values of E_s and E_c under extremely sudden loads are not known with refinement, nevertheless the procedure just outlined is rational and forms an excellent guide to the judgment.

As a second example suppose, in the above case, that the hammer weighs one pound. Following the same procedure it will be found that, neglecting air resistance, a fall of about 18000 ft. would be required, which would take about 35 sec., and that the velocity of the projectile would be more than 1000 ft./sec. Of course such an event would be disastrous both to concrete and projectile—the effect on the reinforcement is problematic.

Shocks may be of any degree of violence. A stick of dynamite exploded while lying across a railroad rail will cut off that rail. A sledge hammer which misses its mark and strikes the face of an anvil —both are extremely violent. On the other hand shock is present even in the slow motion of a testing machine, for if the machine is stopped the load indicated by the scale beam immediately drops off. The "suddenly applied load" of the texts is somewhere in between.

In buildings and bridges we are concerned with such impact as does not damage the concrete, and hence must confine our thought to falls of at most a few feet. It is surprising how low the fall when permanent deformations of both beam and hammer are first manifested. Figures taken from Example 1 will illustrate, as follows:

f_s under static load of hammer = 11400 p.s.i.
f_s, load "suddenly applied" = 22800 p.s.i.
f_s, one-eighth inch drop = 45000 p.s.i.

Even were the approximate nature of the application of Eq. 4B to reinforced concrete to result in 100% error, the drop is still a small fraction of an inch. The impact load which will prove most damaging to the reinforcement in a concrete structure, therefore, is one caused by a weight at least as great as the member affected, falling through a very short distance. It is hard to conceive of such a load in a building.

The foregoing study is, of course, applicable only when the elastic limits of the component materials are not exceeded. The remarkable reserve strength before collapse under impact is brought out in Part III.

Part III

Two series of beams were tested, the first with a hammer weighing 560 lb. and the second with a hammer weighing 2040 lb. The first hammer was the heaviest available at the time the first series was tested, and the second was the heaviest which could be handled with our apparatus.

Concrete Mixes. Lehigh portland cement was used in all tests. The coarse aggregate was nominal ¾-in. Delaware River gravel having a fineness modulus of 5.41, and the fine aggregate was a Delaware River sand with a fineness modulus of 2.93. Both sand and gravel were composed of clean, hard grains with a very small percentage of dirt.

In a few preliminary beams concrete with a cylinder strength of 6000 p.s.i. was tried, but the resulting beams were no stronger than those of 3000 lb. concrete. For the remainder of the program 3000 lb. concrete was used.

Reinforcement. The majority of the beams were reinforced with round bars of either structural or rail steel. A few beams of Series I were reinforced with a silico-manganese billet steel having an extra-high yield point. This steel contained C = 0.54 per cent, Si = 1.96 per cent, and Mn = .86 per cent, thus falling under classification SAE 9250. A few others of Series I were reinforced with another hard grade billet steel having a yield point of 65,400 p. s. i. Owing to the small size of the bars, yield points were higher than

normal for larger bars. The average values are shown in Tables 1 and 2. The ¼ in. bars of structural grade were smooth, all others being deformed.

Test Beams. Nearly all the beams were 10 in. wide and 16 in. deep. A few were 10 in. wide and 8 in. deep. In all deep beams the reinforcement was supported on 1-in. blocks so as to give 1 in. full coverage. In the shallow beams the reinforcement had ½ in. coverage.

In an effort to avoid masking the results by introducing other elastic elements into the beams, no web reinforcement was provided. All beams were 9 ft. long and were tested on an 8 ft. span.

In the top of each beam was embedded a ¾ in. steel plate 6 in. square to prevent local shattering under the blows.

The beams were handled by means of lifting hooks—pieces of ¼ in. round bars shaped like hairpins—embedded about 6 in. into the upper surface of the beams at the outer quarter points of the span.

TABLE 1—BEAMS 10 IN. BY 16 IN. ON 8-FT. SPAN—SERIES I—560 LB. HAMMER

1	2	3	4	5	6	7	8	9	10	11	12	13
Reinf. Round Bars	%	Steel Stress Due to			Drop Required to Rupture Steel	Drop Required for f_s = 45000[4]	Yield Points			Static Concentrated Load Req'd to Produce Failure, Lbs.[5]		
		Wt. of Beam	Ram Static	Total			Struct.	Rail	Extra High	Struct.	Rail	Extra High
1— ¼"	.03	24700	20100	45100	9 in.[1]	——	45000	86000	112000	680	1930	2720
1— ⅜"	.07	10900	9000	19900	48 in.[2]	.25"	45000	61000	92500	2690	3870	6200
3— ½"	.09	8235	6767	15000	54 in.[3]	.46"	45000	86000	112000	3420	5490	9550
1— ½"	.13	6175	5075	11250	——	.65"	45000	57500	92500	4780	6300	10560
1— ⅝"	.20	3950	3225	7175	——	1.08"	45000	55400	——	7860	9820	——

[1]12-in. blow for the smooth structural steel bars. Smooth bars permitted greater deflection.
[2]46-in. blow for structural steel.
[3]Rail steel. Structural steel a little less.
[4]Computed by means of Equation (4B).
[5]Computed, see "Results."

TABLE 2—BEAMS 10 IN. BY 16 IN. ON 8-FT. SPAN—SERIES II—2400 LB. HAMMER

1	2	3	4	5	6	7	8	9
Reinforced Round Bars	%	Steel Stress Due to			Drop Required to Rupture Steel	Drop Required for f_s = 45000	Yield Point	
		Wt. of Beam	Ram Static	Total			Struct.	Rail
1 — ⅜"	.07	10900	34800	45700	4 in.	——	44800	59200
1 — ½"	.13	6175	19700	25875	28 in.	.0167"	42600	61600
1 — ⅝"	.20	3950	12460	16410	90 in.	.095"	41500	58800

Testing Apparatus. Nearly all impact tests on flexural members have been conducted in machines consisting essentially of a large pendulum, the bob of which strikes the beam horizontally at midspan, the beam lying on its side. Since in our tests a very large impact momentum was desired, the construction of such an apparatus

FIG. 2—GENERAL VIEW OF TESTING FRAME

would have been exceedingly costly. Therefore a drop weight type similar to that used in testing rails was used. The use of this apparatus (Fig. 2) is attended with two serious difficulties. First, the weight remains in contact with the specimen, thus damping its free oscillation. On the other hand, beams in actual structures must withstand the stresses due to their own weight in addition to those that may be caused by shock. Second, in order to determine the breaking load with any degree of exactness, many specimens had to be broken with various heights of fall. As a result the program was somewhat prolonged.

In the testing apparatus the ram slides freely in guides fastened to an A frame which is in turn supported on a concrete base 3 ft. wide, 10 ft. long, and 5 ft. deep. The base weighs approximately 22,500 lb. The beams were supported on the ends of the webs of two 3-ft. sections of 14-in. WF 108 lb. steel beams, 8 ft. center to center, embedded vertically half their length in the concrete base. To prevent the webs from cutting into the concrete, each test beam was bedded in plaster of paris at each end on a ¾ in. by 2 in. steel plate.

The ram was raised by means of a chain hoist and released by a trip hook operated by a cord. The lower surface of the ram was

planed flat and impinged on a 6 by 6 by 2-in. steel block rounded on the upper surface to ensure the central application of the blow. This auxiliary block was placed on top of the steel plate previously embedded in the beam. Fig. 2 is a general view of the testing frame with a beam in place.

Deflection Measurements. The study of the Izod specimens having revealed the fact that the minimum total energy was absorbed when the specimen was broken by a single blow, it was deemed advisable to attempt to break the test beams also by means of a single blow. The results of such a procedure, too, would be of more immediately value to the man engaged in construction, for he is worried primarily about the possibility of the breaking of so-called "brittle" reinforcement.

For this reason it was not possible to take many refined deflection readings. It was impossible in those beams which broke on the application of a single blow, and for the others measurement by means of a foot rule was accurate enough. To make the record more complete, however, to one beam of Series I, reinforced with one ⅝-in. round deformed rail steel bar, a stylus was attached which traced on a smoked glass plate the deflections caused by blows of increasing magnitude. The result is shown in Fig. 3. The deflection under the static weight of the ram is scarcely discernible, while the effect of 2-in., 12-in., and 36-in. blows is prominent. The recovery when the weight was lifted after the 36-in. blow may also be seen.

It should be noted that even after the 2-in. blow a measurable permanent deflection was noticeable. This confirms the conclusion, previously reached, that a low fall is all that is necessary to cause a large steel stress. In fact, from Col. 7 in Table 1 it is seen that a fall of 1.08 in. would cause a steel unit stress of 45000 p.s.i.

Deflection measurements are hard to make for conditions beyond the elastic range of stress. The extreme suddenness of an impact load causes the stylus to vibrate laterally, and permanent elongation of the bottom of the beam (as much as 2 in. in some of our beams) usually results in longitudinal displacement.

Method of Testing. The procedure in both Series was the same. A light blow—2 in. with the light hammer or 1 in. with the heavy hammer—was sufficient to cause the concrete to crack. As may be seen from Tables 1 and 2, a greater blow would have seriously overstressed the steel. It is important, in studying the results, to realize how small an energy load was sufficient to rupture the concrete.

FIG. 3

After the preliminary blow we attempted to strike the beam a blow that would just break it. On a similar beam we tried to deliver a blow that would almost break it, in this manner closely bracketing the breaking load. If the beam was not broken, it was subjected to repeated blows till rupture occurred, if within the capacity of the apparatus.

Main Results. Two surprising phenomena were brought out in the tests. The first was the remarkable insurance against collapse under impact provided even by small amounts of reinforcement. With the light hammer it was found impossible to break one ½-in. round bar, one eighth of one percent reinforcement, in a single 8 ft. 4 in. blow. Repeated blows from this height resulted in increased deflection and shattering of the concrete, and if continued long enough would rip the bar out of the bottom of the beam, as shown in Fig. 4. With the heavy hammer it was just barely possible to break one ⅝-in. round bar. The behavior otherwise, except as mentioned later, was identical.

In column 7 of Tables 1 and 2 are given the drop required to produce a computed stress of 45000 p.s.i. in the reinforcement. For the beams reinforced with one ¼-in. round bar in Table 1 and with one ⅜-in. round bar in Table 2, the values are negative, which is confirmed by comparing the unit stresses in columns 5 and 8. The drop required to cause actual rupture of the steel compared to the drop required to stress the steel to the yield point of structural steel may be visualized by comparing columns 7 and 6. Here again we see the enormous reserve resistance to impact in the reinforcement, before the steel is actually broken.

FIG. 4

This may be visualized in still another way. Suppose that one of the beams had been reinforced with four ⅝-in. round bars (¾ of 1 per cent). Assuming a 6-ft. drop of the 560 lb. hammer, the corresponding computed unit stresses would be 150,000 p. s. i. in the steel and 6020 p.s.i. in the concrete. Yet with this hammer it was impossible to break one ½-in. round bar.

The second outstanding phenomenon was the fact that in our tests no noticeable difference in impact resistance was displayed by any of the grades of steel used, almost identical drops being required

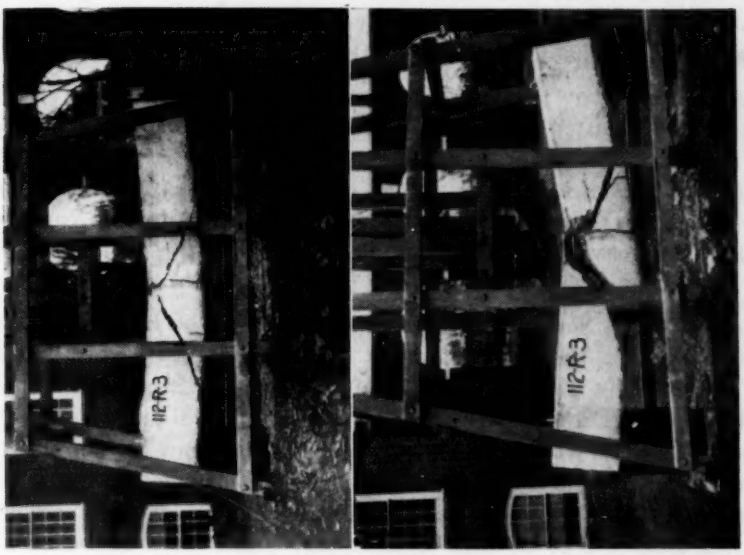

FIG. 7
FIG. 8

FIG. 5
FIG. 6

to break bars of a given size no matter of what the grade steel. The beams reinforced with structural steel deflected more than the others, probably thus absorbing the energy that beams reinforced with stronger steels withstood by direct stress.

Figs. 5 and 6 show a beam reinforced with one ½-in. round structural grade bar after the first and second full height blows of the light hammer. These may be compared with Figs. 7 and 8, which are similar views of a beam reinforced with one ½-in. round rail steel bar. With larger bars, at energy loads less than required to rupture the bars, the heavier hammer produced failures of the same appearance.

Other Results. Several other interesting points of behavior were observed. First, the light hammer often bounced once or twice after striking the more heavily reinforced beams from a medium height. The period of oscillation was estimated at about one-fourth of a second. This would indicate an elastic action not contemplated by the preceding analysis, but since it did not occur within the dangerous range the phenomenon is merely noted. The vertical cracks extending from top to bottom of the beam which often appeared at the outer quarter points of the beams tested with the light hammer are believed to have been caused by this oscillation.

Conclusions. The foregoing tests lead to the following conclusions.

(1) Under impact loads of as extreme a character as we could devise, and probably far more extreme than likely to happen to an actual structure, it was impossible to break over one-fifth of one per cent of reinforcement, even with a hammer weighing nearly twice as much as the beam. Hence even small amounts of reinforcement may be regarded as a good insurance against failure from shock.

(2) The factor of safety against rupture of the reinforcement is very great.

(3) The effect of yield in the supporting members in buildings, and the cushioning effect of tires and springs on bridges, will give an even greater degree of safety.

(4) Rail-steel and other "brittle" steel reinforcements are quite as resistant to impact as is structural steel reinforcement.

Discussion, to close in the September Supplement, should reach A. C. I. Secretary by July 20.

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(Supplement)

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AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN *SEPTEMBER 1940*

Discussion of a paper by T. D. Mylrea:

Effect of Impact on Reinforced Concrete Beams*

BY ARTHUR R. LORD, T. D. MYLREA, J. R. SHANK, M. O. WITHEY, N. H. WITHEY AND H. J. GILKEY

CONVENTION DISCUSSION

Arthur R. Lord, presiding officer—I will open the discussion by asking one question: I should like to know whether, after you got through your tests, you thought these beams were unduly stubborn or that your formula was off or why did they take so much more load?

Professor Mylrea—Largely because we know there is a great discrepancy between reinforcing steel when it reaches the elastic limit and when it actually fails. The elongation during the plastic stage absorbs a great amount of energy.

J. R. Shank†—I believe that a study of the report of the explosion during the construction of the State Office Building in Columbus will bear out Professor Mylrea's results. Considerable rail steel was used in this building. After a large gas explosion there was no evidence of steel breaking off short. I mention that by way of corroboration.

M. O. Withey‡—I should like to ask Professor Mylrea what he knows about the diagonal tensile stresses developed in those beams? It appears to me that the failure in those impact tests is one of diagonal tension in the concrete.

Professor Mylrea—That is an open question. I was trying to find exactly how serious a blow it would take to break the steel. A phenomenon which I did not mention in the paper was this: if I had struck the beam as much as a quarter inch off center, diagonal failure would have followed inevitably. One of those diagonal cracks went

*JOURNAL, Amer. Concrete Inst., June 1940; *Proceedings* Vol. 36, p. 581.
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off like that and let the beam down. I had to be very careful to strike the beam exactly in the center or I would rupture the steel. To reinforce the steel with stirrups and bent bars would have introduced so many other elements that I deliberately left them out and took a great deal of care with the application of the blow.

*N. H. Withey**—I would like to ask whether or not the conclusions from this study mean that we should not consider the impact resistance of highways and bridges and other structures where we know that we get tremendous impact loads? This test indicates that we cannot actually break the steel by any normal impact load. But if we crack the concrete seriously we are going to injure the structure so that it may let in water to expand on freezing and thawing and make an unsightly appearance, or cause other damage that will be rather serious.

Professor Mylrea—One of the fundamental assumptions made in the design of reinforced concrete is that the concrete will be cracked. Yet for a good many years the question has arisen whether or not a cracked beam has failed. I think not. Where water may penetrate and cause rust we must design members so that cracks will not occur, but in the ordinary beam, as designed, we anticipate the cracking of the concrete. Consequently, in these tests, I applied light loads at first for the purpose of cracking the concrete, so that we might compare the stresses developed by the blow under the assumed conditions.

H. J. Gilkey†—I think one other point might be mentioned. There is so much energy to be absorbed when the weight falls; that energy will be absorbed by the weight and by the supports, which, in this case did not absorb very much energy because they took very little deformation, and lots of energy is absorbed by the concrete in the beam. Whatever energy gets to the steel is absorbed there. It is impossible to discriminate between the energy taken by the concrete and that taken by the reinforcing steel. I believe the reason for this investigation was primarily to ascertain the extent to which a high elastic limit steel or brittle steel was a source of vulnerability to impact in concrete. This was to be done by punishing a reinforced concrete beam severely. Apparently the concrete went to pieces very badly before the steel could be broken. In other words, even a brittle steel was virtually packed in an excelsior of concrete and punishment could not get to the steel in a way to be serious.

Professor Mylrea—That is exactly the situation.

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Revised Application of Fineness Modulus in Concrete Proportioning*

By Henry L. Kennedy†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

This paper presents a method of concrete proportioning in which trial mixes are used only for the purpose of final adjustment of mix parts. Fineness modulus is used only as a measure of effective fineness of combined aggregate to facilitate testing and control. Actual tests for compressive strength or other properties are used as a reliable check of the mix design.

Preference is given to the use of water cement ratio as the first step in the design. When the fineness of combined aggregate is expressed in terms of fineness modulus, instead of as gravel-sand ratio, a single determination of optimum fineness serves for all reasonable gradations of fine or coarse aggregates of a given type and maximum size. Optimum fineness is determined from compression tests on a series of concretes of fixed cement content and substantially equal workability. The concrete in this series which gives maximum strength seems to define the coarsest and most economical mix from which fully satisfactory concrete can be made. Charts are provided for use only in selecting fineness modulus where the making of compression tests is not feasible prior to selection of proportions.

INTRODUCTION

The purpose of the present paper is to describe some of the tests which were made in an attempt to determine the optimum fineness modulus for certain specified materials. In addition, a straightforward method of designing mixes is described, using the results of these tests as a guide. The method takes cognizance of size and type of aggregate as well as richness of mix. It will be shown that these factors must be considered to obtain the best proportions of aggregate.

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Probably the most important step in the design of a concrete mix is to determine the proportion of fine to coarse aggregate. Yet there is no one method in use for this purpose. Concrete technicians in various parts of the country have different methods and often arrive at quite different proportions. Where the emphasis is on economy, the tendency is toward under-sanded and harsh mixes; but where workability is the first consideration, the tendency is toward over-sanded mixes.

The single index, known as "fineness modulus" and originally conceived by Duff Abrams, appears to be the best measure of the effective fineness of an aggregate. The fineness modulus is computed from a sieve test simply by adding the percentages coarser than each of certain specified sieves. Due to the judicious choice of sieves, this simple addition automatically makes allowance for the fact that finer sizes have more surface area and add more to workability. The remarkable result is that for a given type and amount of cement and aggregate, all mixes having the same fineness modulus of combined aggregate have practically the same water requirement for a given workability.

Here is where the concrete technician is apt to take a false step. He may be led to believe that fineness modulus is the "missing link" that will enable all important features of a mix to be taken from conceivable charts or tables without recourse to trial. The fallacy is in attempting to obtain a precise relation between water ratio or strength on the one hand and cement content or mix proportions on the other. There are too many extraneous variables. Besides fineness modulus of aggregate and consistency of fresh concrete, the exact water cement ratio and strength of a concrete of specific proportions will depend upon (1) fineness and composition of cement, (2) roughness and shape of aggregate particles, (3) temperature of fresh concrete and probably other factors.

It should be recognized at the outset that when a particular water cement ratio or strength is specified, the cement content or corresponding mix parts cannot be determined without recourse to trial or experience with the materials involved. Tables and charts may serve for a rough approximation but trial mixes are necessary for final adjustment.

The fineness modulus serves its purpose best when used only as a measure of the effective, or average, fineness of an aggregate. All engineers agree that for any concrete there is some average fineness of combined aggregate that is better than any other. The most desirable way to express that average fineness appears to be by fineness modulus.

Suppose, for instance, that by some time-consuming procedure the best possible aggregate proportions were determined for a particular set of conditions and materials. There would then be the alternative of expressing the average fineness represented by these proportions as a certain ratio of fine to coarse aggregate or as a certain fineness modulus. A given ratio of fine to coarse aggregate is equal to a certain fineness modulus only so long as the sand and coarse aggregate do not vary in gradation. But suppose that from day to day the individual aggregates vary in grading as they do on any job. A new ratio of fine to coarse aggregate will be "best" for every change and a new time-consuming test might be needed to determine the optimum ratio again and again.

When the best average fineness of combined aggregate is expressed in terms of fineness modulus, that best fineness can be maintained continually without repeating the time-consuming test. It is then possible on the basis of sieve tests only, to revise the ratio of fine to coarse aggregate from time to time, exactly as necessary to maintain the optimum fineness modulus and therefore the best proportions.

TEST PROGRAM

Tests were made to determine the best fineness modulus of aggregate for each of a variety of concretes commonly encountered. For each particular type and maximum size of aggregate, a series of concrete mixes was made in which cement content and consistency were maintained constant, while the fineness modulus of the aggregate was varied by steps. Water cement ratio varied also, because slump was fixed. Under these conditions, the concrete giving highest strengths was assumed to be most desirable, provided its workability was satisfactory.

It was recognized that a procedure of keeping the workability constant would have been preferable to keeping the consistency constant, if a satisfactory measure of workability were available. Comparative workability was estimated by the ease of rodding the green mix. Actually the concrete giving highest strength at constant consistency was not quite as workable as were the more sandy mixes (lower fineness moduli). The difference was slight, however, in the range of economical mixes. On the other hand, the concrete giving highest strength was substantially more workable than were the less sandy mixes. For this reason, and for the reason that the harsh mixes give low strength, fineness moduli higher than the optimum are doubly undesirable.

DETAILS OF TEST

The details of the tests were chosen to include the usual range of ordinary concrete. Both natural and manufactured aggregates were

used in three maximum sizes, ⅜ in., ¾ in. and 1½ in. The cement was one of standard composition having a fineness of 1750 cm. sq. per gm. Four richnesses of mix were used for each maximum size aggregate ranging from a mix of 1-3 by weight to 1-8. Specimens were 4 x 8 in. cylinders for the two smaller sizes of aggregate and 6 x 12 in. where the 1½ in. aggregate was used. Consistency was measured by means of a 12 in. slump cone, the aim being to obtain a 1½ in. slump throughout. Compressive strengths were observed at 28 days. At least three cylinders were tested for each condition, and in several cases tests were repeated to reduce irregularities.

A type of "ideal" gradation of aggregate was used in most of the tests, in order to avoid questions of particle interference and possible effects of poor gradation. Irregular gradations were used in auxiliary tests however, to show the effect of deficiencies of certain sizes. The results of these auxiliary tests will be discussed later.

For the main series of tests the "ideal" gradations used were of the "percentage" type, in which the amount of material retained on each successively smaller sieve is a fixed percentage of that preceding. In other words, a "70 per cent gradation" is one in which there is 70 per cent as much material of the second largest size as of the largest size and again, 70 per cent as much of the third largest sizes as of the second, and so on down. The amount of each size can be found most readily by a simple numerical trial, assuming first an arbitrary weight of 1000 grams for the largest size, and then determining the amounts of successively smaller sizes by multiplying by the proper "percentage" for the gradation in question. The percentage of the various sizes can then be obtained by dividing the amounts by the sum of the weights of all sizes. Complete sieve analyses for any "percentage gradation" commonly encountered can be taken from Fig. 1.

For a given maximum size, a given "percentage" gradation corresponds to a definite fineness modulus. Thus, for a ¾ in. maximum size, an "80 per cent gradation" has a fineness modulus of 4.60, a "70 per cent gradation" has 5.15, and a "60 per cent gradation" has 5.65. When the fineness modulus was to be changed in making a series of tests, it was only necessary to select the corresponding "percentage gradation" then to take from the curves in Fig. 1 the percentages of all sieve sizes to be used.

RESULTS OF TESTS

The principal results of the tests to determine optimum values of fineness modulus for each of a variety of conditions are shown in Fig.

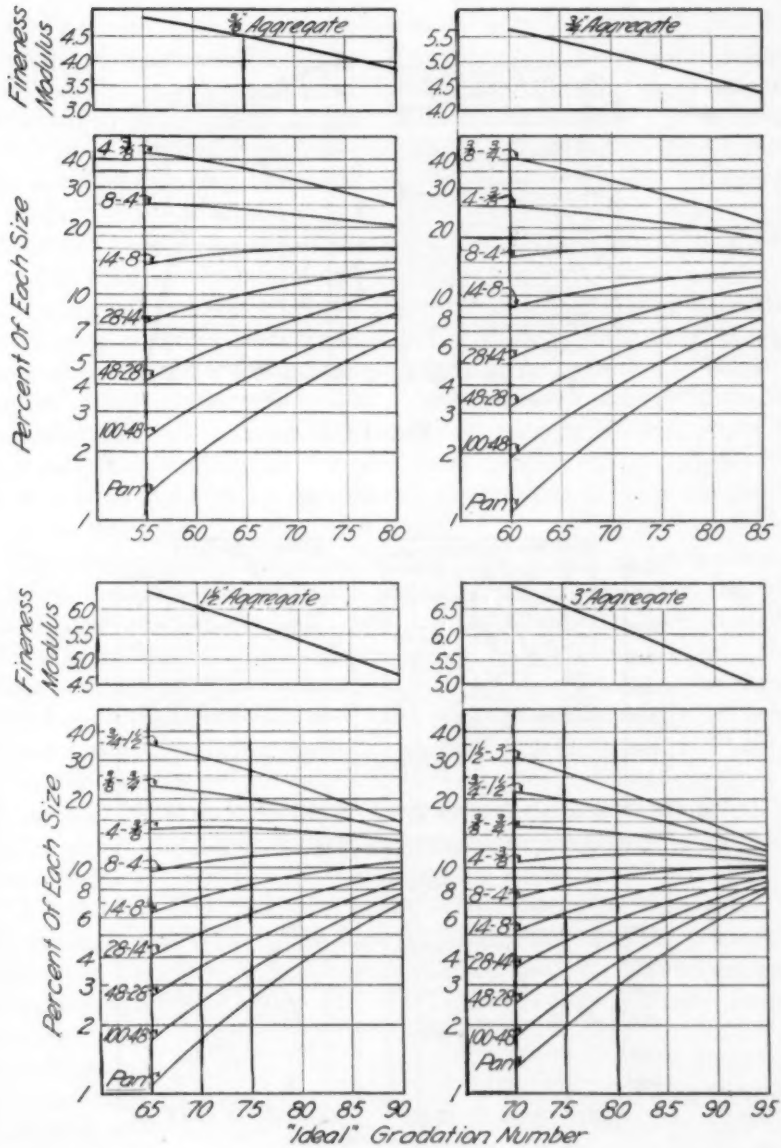


FIG. 1—PERCENTAGE OF INDIVIDUAL SIEVE SIZES TO MAKE "IDEAL" OR "PERCENTAGE" GRADATION FOR VARIOUS FINENESS MODULI. GRADATION NUMBER DEFINES RELATION OF SUCCESSIVE SIEVES SIZES

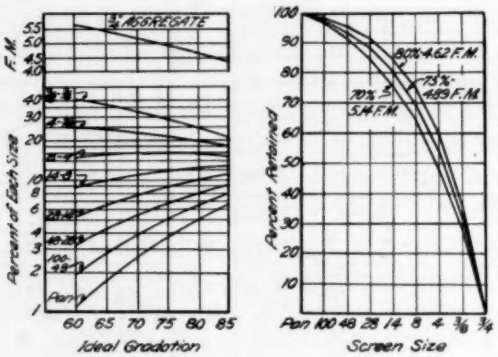


FIG. 1A

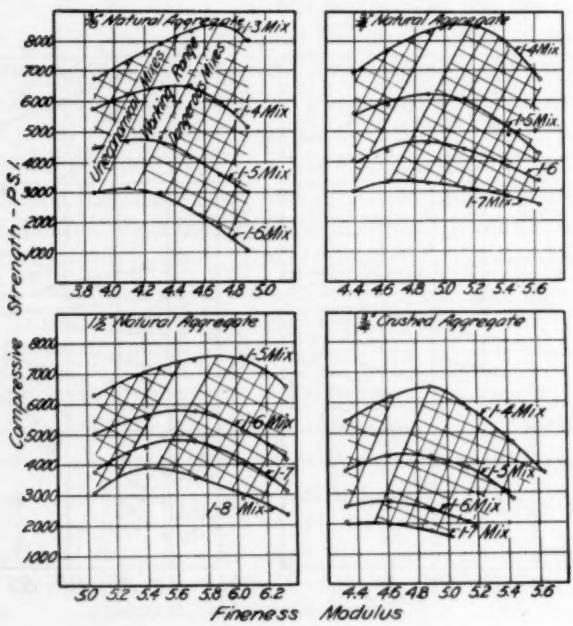


FIG. 2—RESULTS OF TESTS CONDUCTED TO DETERMINE OPTIMUM FINENESS MODULI FOR A VARIETY OF CONDITIONS

2. These diagrams reveal that for each richness of mix and maximum size of aggregate, there is a maximum strength for some one fineness modulus of combined aggregate. The strength is less where the fineness modulus is less (more sand) because of the higher water cement ratio which is necessary to maintain the consistency. The sandy mixes corresponding to points to the left of the maximum strengths in the curves are perfectly satisfactory from every standpoint except economy. They lack economy because to meet a given requirement either as to strength or water cement ratio, more cement would be required than for the mix of optimum fineness modulus.

The strength results of Fig. 2 are especially enlightening when the appearance of the various concretes at the time of mixing is considered. Practically all of the mixes plotted in this figure appeared to be satisfactory when mixed. However, mixes somewhat coarser than the optimum fineness modulus did not develop strengths that might be expected from their relatively low water cement ratios. Hence the areas to the right of the maximum strengths have been shaded to indicate the danger in using such mixes.

The reason for the lower strengths in the case of higher values of fineness modulus must lie in the poorer physical structure of the concrete since most of the water ratios were equal to or lower than for those mixes giving maximum strength. It can hardly be said that there are not sufficient fines to fill the voids, because it can be shown that lowest voids and highest density are obtained when the optimum fineness modulus is exceeded to an appreciable degree. There is probably an increasing number of the larger particles of aggregate in contact with one another as the fineness modulus is increased, but the optimum point must be exceeded by a very appreciable amount before the interference is sufficient to increase the voids.

A band labelled "working range" is indicated in each group of curves in Fig. 2. The upper limit of the band is clear cut because as previously stated, any mix coarser than that giving highest strength is undesirable. The lower limit of the band is less clear cut. Due to the fact that a slight increase in strength may be compensated for by a slight increase in workability, a fineness modulus about 0.2 less than the optimum is substantially as good as the optimum. It is believed, therefore, that the bands marked "working range" include only that range of mixes which are best of all. Actually when job variations are expected to be substantial, the tendency should be to shift toward a still lower fineness modulus.

The disadvantages of using too high a fineness modulus are much greater than using too low a value. One reason for this is the fact that

the oversanded mixes, although uneconomical, do follow the general water-cement ratio curve. Thus there is less opportunity to be misled by the water requirement when designing for strength.

A comparison of the maximum strengths of the different curves of Fig. 2 shows how much the optimum fineness modulus varies with mix or cement content as also with type and size of aggregate. As might be expected, a leaner mix requires a lower fineness modulus. Also, a smaller maximum size of aggregate requires a lower fineness modulus as does a crushed type of aggregate.

The principal results of Fig. 2 are presented in more practical form in Fig. 3. The optimum fineness modulus can be taken from the curves in this figure for any ordinary mix of a maximum size of aggregate up to 1½ in. The points plotted are the midpoints of the "working range" band and represent a decrease from the maximum permissible fineness modulus of 0.1. In practice, the aim should be to use values about 0.1 below those indicated, to allow for variations. The curves apply only for natural aggregate but they apply fairly well also for an aggregate consisting of all crushed material if an additional 0.2 is subtracted from the fineness modulus as shown.

The difference between natural and crushed aggregate is somewhat less than that found by Duff Abrams in his early work. However, natural aggregates with more smooth particles than those used in our investigation would undoubtedly permit coarser gradations to be used for best results and then the difference might more nearly approach Abrams' results. Furthermore, many changes in fineness and composition of cement, which of course effect workability, have been made in the last few years. These too have a bearing on the differences noted between natural and crushed aggregates.

Caution is urged against accepting the curves of Fig. 3 to cover all cases. Use of these curves is advised only as a first approximation or for cases which do not warrant separate determinations of the optimum fineness modulus. On a large number of concrete projects it will be found advantageous to determine the specific optimum fineness modulus for the materials and conditions involved.

Objection might be raised to the use of compressive strength as a measure of the optimum fineness modulus. Although tests of compressive strength seem to reveal the point where a significant change takes place in the quality of concrete, other tests may be substituted to advantage in many cases. If flexural strength is the quality most desired in concrete, then modulus of rupture should be used as a basis

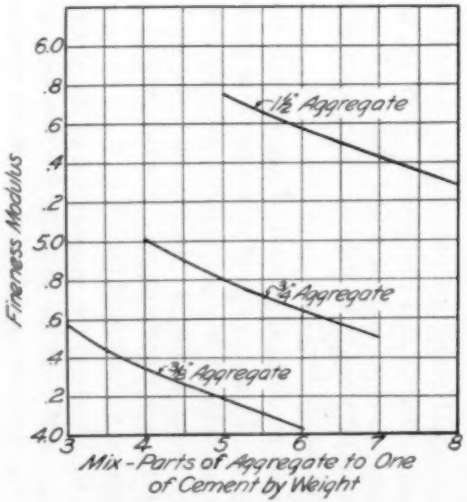


FIG. 3—PRACTICAL FORM OF FIG. 2. FINENESS MODULUS AT OPTIMUM STRENGTH REDUCED BY 0.1

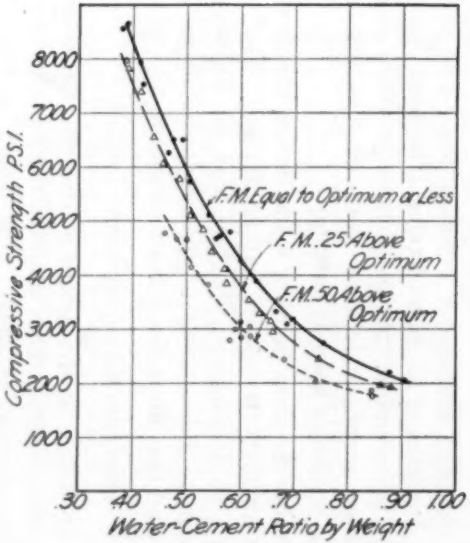


FIG. 4—EFFECT OF W/C ON COMPRESSIVE STRENGTH

for determining optimum fineness modulus. Likewise if there were a simple test for durability such tests might be used instead of compressive strength.

Fig. 4 shows three water cement ratio curves resulting from the strength data obtained during our investigation. The upper curve represents the strengths of all mixes having a fineness modulus equal to or less than the optimum. As the fineness modulus is increased, the curves become progressively lower again emphasizing the danger of exceeding the optimum fineness modulus.

Table 1 contains the data of tests made during this investigation including some auxiliaty tests with gap gradations. These auxiliary tests were made to demonstrate the fact that the fineness modulus method of design is independent of the gradation of an aggregate while the type and maximum size remain constant.

PREFERRED PROCEDURE OF DESIGN

The logical procedure in the design of concrete mixes is to select first the water cement ratio which will give the required strength and durability. The selection of water ratio should be guided by past experience with structures in service, and by results of tests for durability and strength with the particular materials.

After the water cement ratio is selected, a first approximation of the corresponding mix can be taken from the curves of Fig. 5. These curves were plotted from the results of tests described above and cannot apply exactly for all materials and conditions. They are useful however as a convenient means for making a first choice. As an example, if the water cement ratio were to be 0.40 for $\frac{3}{4}$ in. natural aggregate, the approximate mix would be 1:4.2.

The proper fineness modulus of combined aggregate is then chosen by reference to Fig. 3. If large variations in aggregate gradations are expected, the working value of fineness modulus should be set at about 0.2 below the value shown in Fig. 3. In any case, the working value should be at least 0.1 below the optimum. For the present example of a mix 1:4.2 the working value would be 4.85.

The next step is to determine the proportions of the groups fine and coarse aggregate that will yield the working fineness modulus of combined aggregate. The fineness moduli of the separate groups of aggregate must first be determined from actual sieve analyses.

Because of the fact that more than two groups of aggregate can be combined in a variety of proportions to yield a desired fineness modulus, a choice is permitted as to the percentage of all but two sizes.

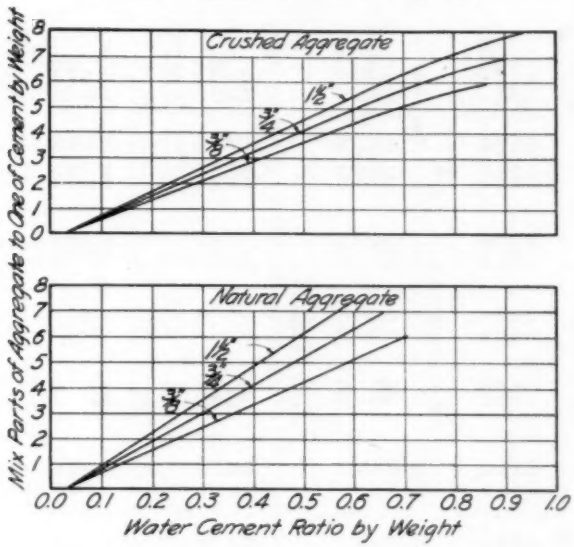

Crushed Aggregate
Natural Aggregate
Mix Parts of Aggregate to One of Cement by Weight
Water Cement Ratio by Weight
0.0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0

FIG. 5—WORKING CURVES FOR ROUGH APPROXIMATION OF MIX PARTS WHEN W/C IS GIVEN

It is then advisable to approximate as nearly as possible some "ideal" grading as well as to obtain the right fineness modulus. For this purpose, the curves of Fig. 1 are useful, because they give ideal gradings corresponding to a wide range of fineness moduli for four different maximum sizes of aggregate. A plot of the percentages of the cumulative sieve sizes as taken from the curves of Fig. 1 for a given fineness modulus and maximum size, when added cumulatively from the coarser sizes downward, can be plotted to provide the complete ideal grading curve. This is illustrated by the example given in Fig. 1A.

The three or more groups of aggregate on the job should then be combined to fit this grading curve as nearly as possible. In order to accomplish this the percentages of intermediate sizes are first chosen to give almost exactly the percentages of sizes indicated by the ideal grading curves, then the percentages of sand and coarsest aggregate can be computed to make the combined fineness modulus correct using the following equation:

$$P_a M_a + P_c M_c + P_b M_b = 100\ F_c\ M_c \text{ (working value)}$$

Where: P_a = selected percentage

P_b = unknown percentage of one group

P_c = unknown percentage of another group which may be represented by $(100\text{-}P_a\text{-}P_c)$

M_a, M_b, M_c = F. M. of respective groups

TABLE 1

NATURAL AGGREGATES—STEP GRADATIONS—3/8 IN. MAXIMUMSIZE

Math. Gradation	F.M.	G/S	W/c	Slump	Strength 28-D**	Wt./c.f. Concrete Cured	Rodability
			MIX BY WEIGHT—1-3				
55%	4.88	0.85	.387	3.0	8000	152.70	Good
60%	4.70	0.70	.380	2.5	8540	152.80	Good
65%	4.50	0.58	.387	3.3	8305	152.30	V. Good
70%	4.30	0.49	.405	2.5	7850	152.00	V. Good
75%	4.08	0.41	.430	2.3	7300	151.70	V. Good
80%	3.86	0.34	.450	2.7	6750	151.20	V. Good
			MIX BY WEIGHT—1-4				
55%	4.88	0.85	.480	2.3	5150	152.80	Poor
60%	4.70	0.70	.486	2.2	5975	152.80	Fair
65%	4.50	0.58	.473	2.7	6500	152.75	Fair
70%	4.30	0.49	.467	2.5	6400	152.30	Good
75%	4.08	0.41	.473	2.7	6200	151.60	Good
80%	3.86	0.34	.495	2.5	5750	151.05	V. Good
			MIX BY WEIGHT—1-5				
55%	4.88	0.85	.590	*	2970	151.20	V. Poor
60%	4.70	0.70	.590	2.3	3550	151.25	Poor
65%	4.50	0.58	.593	2.5	4150	151.20	Fair
70%	4.30	0.49	.580	2.7	4795	151.15	Good
75%	4.08	0.41	.573	2.5	4650	150.50	V. Good
80%	3.86	0.34	.583	2.2	4455	150.10	V. Good
			MIX BY WEIGHT—1-6				
55%	4.88	0.85	.706	*	1095	150.85	V. Poor
60%	4.70	0.70	.686	2.2	1800	150.80	Poor
65%	4.50	0.58	.680	2.0	2425	150.75	Fair
70%	4.30	0.49	.660	2.0	2950	150.55	Fair
75%	4.08	0.41	.686	2.5	3100	149.90	Good
80%	3.86	0.34	.706	2.5	2970	149.25	Good
							W1386

NATURAL AGGREGATES—STEP GRADATIONS—3/4 IN. MAXIMUM SIZE

			MIX BY WEIGHT—1-4				
60%	5.64	1.86	.405	2.0	6750	154.60	Poor
65%	5.43	1.53	.391	2.5	7825	155.00	Good
70%	5.14	1.16	.391	2.5	8640	154.50	Good
75%	4.89	0.95	.404	2.8	8330	154.35	V. Good
80%	4.62	0.77	.420	2.8	7700	153.50	V. Good
84%	4.39	0.65	.440	2.8	6960	152.90	V. Good

MANUFACTURED AGGREGATES—STEP GRADATIONS—3/8 IN. MAXIMUM SIZE

Math. Gradation	F.M.	G/S	W/c	Slump	Strength 28-D**	Wt./c.f. Concrete Cured	Rodability
			MIX BY WEIGHT—1-3				
60%	4.70	0.70	.413	2.0	7400	149.90	Fair
65%	4.50	0.58	.413	2.0	7930	149.40	Good
70%	4.30	0.49	.433	2.3	7650	148.80	Good
75%	4.08	0.41	.454	2.5	6840	147.90	V. Good
			MIX BY WEIGHT—1-4				
65%	4.50	0.58	.500	3.0	4680	146.50	Fair
70%	4.30	0.49	.530	2.5	4840	145.50	V. Good
75%	4.08	0.41	.540	2.5	5100	145.90	V. Good
80%	3.86	0.34	.565	2.7	4860	145.00	V. Good
			MIX BY WEIGHT—1-5				
70%	4.30	0.49	.617	2.7	2875	145.10	Poor
75%	4.08	0.41	.655	2.5	3150	144.80	Fair
80%	3.86	0.34	.693	2.5	3190	144.00	Good
85%	3.64	0.28	.713	2.3	2970	143.00	Good
			MIX BY WEIGHT—1-6				
70%	4.30	0.49	.845	2.0	1880	144.50	Fair
75%	4.08	0.41	.859	2.3	1960	143.90	Fair
80%	3.86	0.34	.879	2.0	2200	144.30	Good
85%	3.64	0.28	.920	2.2	2000	142.50	Good
							W1354

MANUFACTURED AGGREGATES—3/4 IN. MAXIMUM SIZE

			MIX BY WEIGHT—1-4				
60%	5.64	1.86	.473	*	3650	149.85	V. Poor
65%	5.43	1.53	.480	2.5	4680	150.40	Poor
70%	5.14	1.16	.486	2.3	5780	150.50	Fair
75%	4.89	0.95	.495	2.7	6500	149.80	Good
80%	4.62	0.77	.500	2.3	6150	148.70	Good
85%	4.33	0.62	.530	2.3	5400	158.40	Good

(TABLE 1—CONTINUED)

MIX BY WEIGHT—1-5

60%	5.64	1.85	.466	*	4250	154.70	Poor
65%	5.43	1.53	.460	1.8	4780	154.90	Poor
70%	5.14	1.16	.457	2.5	6050	155.00	Fair
75%	4.89	0.95	.467	2.5	6250	153.95	Good
80%	4.62	0.77	.500	2.7	5900	153.00	V. Good
84%	4.39	0.65	.526	3.3	5620	151.90	V. Good

MIX BY WEIGHT—1-6

60%	5.64	1.86	.507	*	3365	153.75	V. Poor
65%	5.43	1.53	.540	1.5	3810	154.00	Poor
70%	5.14	1.16	.547	2.5	4410	153.65	Fair
75%	4.89	0.95	.554	3.0	4660	153.65	Good
80%	4.62	0.77	.585	2.8	4350	153.15	V. Good
84%	4.39	0.65	.620	3.0	4050	151.45	V. Good

MIX BY WEIGHT—1-7

60%	5.64	1.86	.606	*	2510	152.40	V. Poor
65%	5.43	1.53	.606	*	2840	153.50	V. Poor
70%	5.14	1.16	.617	2.2	3050	153.90	Poor
75%	4.89	0.95	.635	2.5	3290	152.85	Fair
80%	4.62	0.77	.665	2.2	3310	152.45	Good
84%	4.39	0.65	.687	2.2	2990	151.15	V. Good
							W1351

NATURAL AGGREGATES—STEP GRADATIONS—1½ IN. MAXIMUM SIZE

MIX BY WEIGHT—1-5

65%	6.34	2.86	.440	2.7	6550	156.65	Fair
70%	6.05	2.18	.420	2.5	7510	156.00	Good
75%	5.73	1.67	.434	2.7	7445	155.85	Good
80%	5.40	1.29	.452	2.5	7050	154.50	Good
85%	5.05	1.01	.466	2.3	6275	153.95	Good

MIX BY WEIGHT—1-6

65%	6.34	2.86	.510	2.5	4150	155.85	Fair
70%	6.05	2.18	.506	2.3	5100	155.20	Fair
75%	5.73	1.67	.506	2.7	5725	155.00	Good
80%	5.40	1.29	.513	2.3	5550	154.10	Good
85%	5.05	1.01	.537	2.0	5000	153.40	Good

MIX BY WEIGHT—1-7

65%	6.34	2.86	.600	*	3125	155.60	Poor
70%	6.05	2.18	.570	2.5	4100	155.05	Fair
75%	5.73	1.67	.560	2.3	4710	155.05	Good
80%	5.40	1.29	.560	2.5	4560	154.60	Good
85%	5.05	1.01	.686	2.0	3755	152.40	Good

MIX BY WEIGHT—1-8

65%	6.34	2.86	.600	*	2300	155.55	V. Poor
70%	6.05	2.18	.600	*	2860	155.50	Poor
75%	5.73	1.67	.614	2.8	3525	154.20	Fair
80%	5.40	1.29	.626	2.0	3890	153.75	Good
85%	5.05	1.01	.700	2.8	3040	152.35	Good
							W1352

MIX BY WEIGHT—1-5

65%	5.43	1.53	.580	*	2790	148.80	Poor
70%	5.14	1.16	.574	2.0	3845	149.95	Fair
75%	4.89	0.95	.600	2.5	4250	149.20	Good
80%	4.62	0.77	.633	2.7	4230	147.80	Good
85%	4.33	0.62	.666	2.3	3740	147.60	Good

MIX BY WEIGHT—1-6

70%	5.14	1.16	.740	2.0	2025	147.50	Poor
75%	4.89	0.95	.746	2.7	2420	148.00	Fair
80%	4.62	0.77	.753	2.5	2725	147.30	Good
85%	4.33	0.62	.786	2.3	2570	145.00	V. Good

MIX BY WEIGHT—1-7

75%	4.89	0.95	.845	2.0	1775	146.80	Fair
80%	4.62	0.77	.880	2.3	1930	146.20	Good
85%	4.33	0.62	.905	2.3	2040	145.10	Good
90%	4.05	0.50	.980	2.5	2000	144.25	V. Good
							W1356

NATURAL AGGREGATES—GAP GRADATIONS—¾ IN. MAXIMUM SIZE

MIX BY WEIGHT—1-6

GAP 28-14

68%	5.43	1.55	.550	2.0	3770	155.75	Fair
78%	4.91	1.00	.554	2.5	4940	154.30	Good
88%	4.31	0.66	.606	2.3	4325	151.75	Good

GAPS 4⅜, 14-8, 48-28, -100

75%	5.32	0.95	.540	2.5	4190	155.25	Poor
83%	4.90	0.67	.554	2.3	4690	153.40	Good
93%	4.36	0.44	.653	2.3	3610	151.20	Good
							W1385

*Indicates that cone broke when form was removed making it impossible to get a correct slump measurement.

**Each result an average of four 4 x 8 cylinders. W/c is by weight.

In the example, if the fineness moduli of the groups of coarse and fine aggregate were 7.03, 6.16 and 2.85 respectively and the selected percent of intermediate gravel indicated by the ideal grading curve was .18, the percent of sand and coarse aggregate would be,

$$(.18 \times 6.16) + (P_b \times 7.03) + (1.00 - .18 - P_b)\ 2.85 = 4.85$$

$$P_b = 33.5$$

$$P_c = 48.5$$

Selected $P_a = 18.0$

Total 100.0

The approximate mix would then be 1:2.04:0.75:1:41.

A trial mix of these proportions should then be made as a check to determine the adjustment necessary to yield the desired water cement ratio. It should not be expected that the mix would be exactly right without adjustment, because different cements and aggregates require different amounts of a water. If the trial mix gives a slump too high or too low at the desired water cement ratio, the adjustment can readily be estimated from the fact that the slump is increased one inch by a decrease in combined aggregate parts of 0.25 and vice versa.

After this adjustment, the best mix is presumably determined. It is highly advantageous, however, to make compressive strength tests

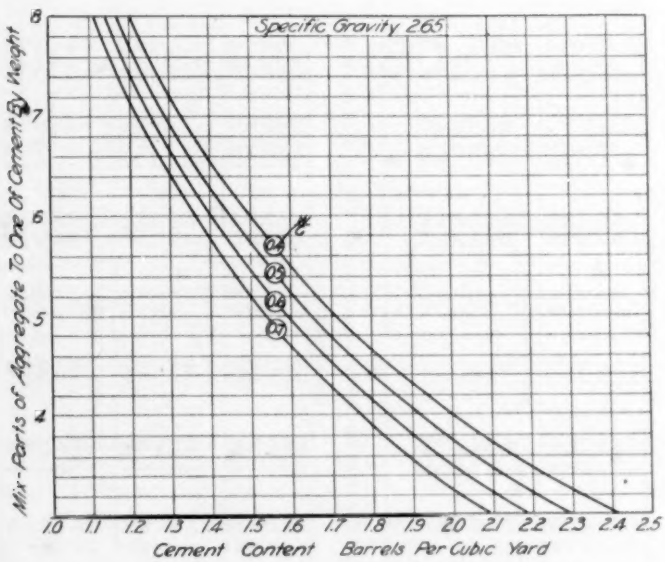


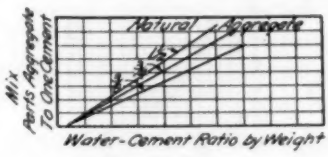
FIG. 6—WORKING CURVES FOR APPROXIMATION OF MIX PARTS WHEN CEMENT CONTENT IS GIVEN

CASE I

GIVEN: Water-cement ratio, maximum size of aggregate and slump

A.

Determine Approximate Mix Parts



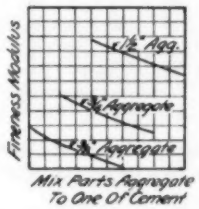
For Rough Approximation of Trial Mix Only

Correction factor: Increase or decrease mix parts .25 for each inch decrease or increase in slump from from 2.5 inches.

KNOWN: Approximate mix parts, maximum size and type of aggregate

B.

Select indicated optimum F. M.



KNOWN: Total approximate mix parts and indicated optimum F. M., two groups of aggregate and F. M. of each

C.

Determine percentage of each group to be used.

Substitute the F. M. in the following equation where P equals the per cent of each group and M the corresponding F. M. Solve for P_a abd P_b.

$$P_a M_a - (100 - P_a) M_b = \text{F. M. optimum}$$

Repeat for F. M. .50 and .25 above and .25 below selected value.

D.

Make one trial mix at optimum F. M. to determine slump. Adjust mix using correct factor to obtain desired slump.

E.

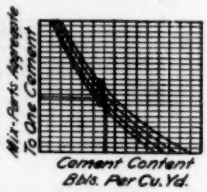
Make three trial mixes to desired slump, one below and two above optimum. Cast several cylinders of each mix and break at 7 or 28 days. Plot compressive strength vs. F. M.

FIG. 7—TYPICAL FLOW SHEET FOR USE WITH TWO GROUPS OF AGGREGATE WHEN W/C IS GIVEN

CASE 2

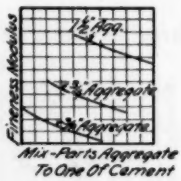
GIVEN: Cement content or aggregate parts

A.
Determine mix parts or cement content



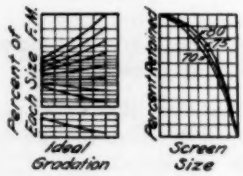
KNOWN: Mix parts, maximum size and type of aggregate

B.
Select indicated optimum fineness modulus



KNOWN: Three or more groups of aggregate, sieve analysis and F. M. of each, also total mix parts and indicated optimum F. M.

C.
Determine ideal gradation



1. Determine the ideal gradation for the selected F. M. (upper curve Fig. x), also for a F. M. .25 above and .25 below this value.
2. Determine sieve analysis for these ideal gradations (lower curve Fig. x). Plot these as Fig. y.
3. From Fig. y determine per cent of material required for the size of greatest preponderance as shown by the sieve analysis of the given aggregates.
4. Substitute this percentage for P_a in the following equation where P is equal to the percentages of the various groups of aggregate and M is equal to the respective F. M. Solve for P_b and P_c.

$$P_aM_a + P_bM_b + (100 - P_a - P_b)M_c - \text{F. M. optimum}$$

D.
Make three trial mixes to identical slump. Cast several cylinders each and break all at 7 or 28 days. Plot compressive strength vs. F. M.

FIG. 8—TYPICAL FLOW SHEET FOR USE WITH THREE OR MORE GROUPS OF AGGREGATE WHEN CEMENT CONTENT IS GIVEN

to check the optimum fineness modulus for the materials involved. It is suggested that at least three mixes be made in addition to the indicated optimum, namely, two coarser and one finer than the indicated optimum value. Several specimens of each mix should be tested either at 7 or 28 days. If the results of these tests show an advantage in a slight change of combined fineness modulus, corresponding slight changes in mix can be made to obtain the final mix.

Note: If only two groups of aggregates are involved, no choice of the percentage of either group is possible since the percentage of each group is solved for directly by the following simple equation:

$$P_a M_a + (100 - P_a) M_c = 100 \text{ F. M}_b \text{ (working value)}$$

ALTERNATE PROCEDURE

Where cement content and limiting water ratios are given, Fig. 6 may be used to advantage. These curves were plotted for four water cement ratios, namely, 0.4, 0.5, 0.6 and 0.7 from calculated values based on the absolute volume method using a specific gravity of 2.65.

As an example if the cement content were 1.4 bbl. per cu. yd. with a water ratio of 0.5 the mix would be 1-6.3.

Having determined the mix parts, the preferred procedure is followed except that the adjustment of mix parts is unnecessary.

Note: A further graphical check may be made by mathematically combining the groups of aggregate of the designed mix. The percentage of each sieve size is plotted and the resulting curve compared with the ideal gradations of the selected fineness modulus.

CONCLUSIONS

The design procedure is presented in the form of two flow sheets Fig. 7 and 8 (Case 1 and 2) and cover conditions of design when water cement ratio is specified, and when cement content is specified, or mix parts are specified although the former is preferred.

These flow sheets emphasize the simplicity of the design method despite the fact that short cuts are avoided. The few trial mixes provide a reliable check of the mix design. This method is straightforward and results in a mix design which seems to eliminate the personal equation to the greatest degree.

Discussion, to close in the September Supplement, should reach A. C. I. Secretary by July 20.

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Discussion of a paper by Henry L. Kennedy:

Revised Application of Fineness Modulus in Concrete Proportioning*

BY C. A. G. WEYMOUTH, W. H. PRICE AND AUTHOR

BY C. A. G. WEYMOUTH†

Mr. Kennedy has performed a real service in restating the limits for fineness modulus in terms of proportions by weight. His treatment of the subject is excellently presented and the use of his mathematical curves in his tests makes for simplication.

The writer has made a study of Mr. Kennedy's data to discover the influence of the spacing of the particles of the dominant size-group upon the optimum fineness modulus. A direct correlation can be traced in all the tests, thus showing the importance of the author's intention "to avoid questions of particle interference and possible effects of poor gradation."

In his early work, the writer set up the criterion for particle interference as that spacing between particles of one size-group that would just permit free movement between them of the particles of the next smaller group-size in the aggregate, denoted as $t = D_1$. This criterion always gave excellent workability in field concrete and even for the more difficult placing against waste mold work. Recently he has re-examined a large amount of late data where the workability of each mix is carefully evaluated and has found that many observers are satisfied with the workability in mixes where the critical spacing is somewhat closer than $t = D_1$. Mr. Kennedy's data provides a reasonable explanation for this.

*JOURNAL, Amer. Concrete Inst., June 1940.
†Concrete Engineer, Raymond G. Osborne Laboratories, Los Angeles.

The spacing of particles of a size-group in a concrete mix is measured by their concentration, or relative density. In all of the concretes of Table 1 of the paper, the grading curves of the combined aggregates are such as to put the greatest concentration in the size-group of each mix having the maximum diameter. The relative density of this dominant size-group, therefore, becomes equal to the absolute volume per unit volume of mix of the material of largest size. Table A, accompanying, contains the results of the writer's computations for the mixes with the natural aggregates which fall in the "working band" shown by Mr. Kennedy. It will be seen that values of the relative density for the concretes of maximum strengths vary from 0.247 to 0.171 for continuous grading. The rodability of these is marked in Table 1 as "fair" or "good" while the mixes with the next smaller fineness moduli are considered more workable, being marked as "good" or "very good." In general, increase in the clear distance between the dominant particles provides increased workability.

For rounded gravel, the spacing, $t = D_1$, is obtained with a relative density of 0.195 for continuous grading and 0.333 for a jump grading of one gap. The last column of Table A gives the ratios of the actual relative densities to these theoretic values.

The ease of rodding depends upon three things: (1) the maximum-size particles in the mix. Three-eighths inch pieces are more easily stirred or pushed aside than larger pieces. (2) The lubricating effect of cement paste. Mixes with the greatest cement contents permit high relative densities for equal rodability. (3) The richness of the mortar, or sand-cement ratio as measured by a/c. For the same cement contents, richer mortars permit larger values of relative density.

The low ratio of 82 per cent obtained for the gap grading is typical and in agreement with observations by other investigators. Jump gradings possess a common fault; they are deficient in cohesion and cannot properly support the large sizes; these settle in the mix, become concentrated, and this causes local particle interference even if the dominant group is limited to the theoretic amount.

Considering these three reasons for variations in the critical spacing of the largest particles, it is evident from a study of Table A that the relative density of the dominant size-group plays a most important role in determining the value of the optimum fineness modulus and the optimum compressive strength of concrete.

The writer is pleased to find that these data confirm his announcement made several years ago, that particle interference tends to

decrease compressive strength for a given water-cement ratio. Mr. Kennedy shows this by his Fig. 4.

TABLE A—OPTIMUM FINENESS MODULUS COMPARED WITH RELATIVE DENSITY OF THE DOMINANT SIZE-GROUP

Data from Harry L. Kennedy's Table 1.
Computations for mixes giving maximum compressive strengths and next smaller fineness moduli.
Specific gravity taken as 2.65.
Air content assumed as 15 per cent of water volume.
Relative density of 0.195 gives spacing: $t = D_1$ for rounded gravel with continuous grading. For a gap of one group, this value becomes 0.333.

Math. Gradation	Mix	a/c	F.M.	F.M. Sand	Sand %	Sacks per Yard	b	No. 4 to ⅜" Abs. Volumes	⅜" to ¾" Abs. Volumes	¾" to 1½" Abs. Volumes	Relative Dens. of Dominant Group: Value	Relative Dens. of Dominant Group: Ratio
⅜-Inch Maximum												*
60%	1—3	2.09	4.70	3.79	58.8	9.50	.247	.247	.000	.000	.247	127%
65%	1—3	2.32	4.50	3.63	63.3	9.45	.219	.219	.000	.000	.219	112%
70%	1—3	2.39	4.30	3.46	67.1	9.38	.194	.194	.000	.000	.194	100%
65%	1—4	3.00	4.50	3.63	63.3	7.56	.234	.234	.000	.000	.234	120%
70%	1—4	3.18	4.30	3.46	67.1	7.57	.210	.210	.000	.000	.210	108%
75%	1—4	3.40	4.08	3.33	71.8	7.56	.178	.178	.000	.000	.178	91%
70%	1—5	3.98	4.30	3.46	67.1	6.24	.216	.216	.000	.000	.216	111%
75%	1—5	4.26	4.08	3.33	71.8	6.25	.194	.194	.000	.000	.194	100%
75%	1—6	5.10	4.08	3.33	71.8	5.32	.189	.189	.000	.000	.189	97%
80%	1—6	5.30	3.86	3.13	74.6	5.28	.169	.169	.000	.000	.169	87%
¾-Inch Maximum												
70%	1—4	2.19	5.14	3.45	46.3	7.87	.356	.147	.209	.000	.209	107%
75%	1—4	2.43	4.89	3.29	51.3	7.79	.279	.119	.160	.000	.160	82%
75%	1—5	3.04	4.89	3.29	51.3	6.53	.335	.144	.191	.000	.191	98%
80%	1—5	3.35	4.62	3.13	56.5	6.44	.295	.131	.164	.000	.164	84%
75%	1—6	3.64	4.89	3.29	51.3	5.56	.342	.147	.195	.000	.195	100%
80%	1—6	4.02	4.62	3.13	56.5	5.38	.296	.132	.164	.000	.164	84%
80%	1—7	4.68	4.62	3.13	56.5	4.81	.308	.137	.171	.000	.171	88%
84%	1—7	5.02	4.39	2.99	60.6	4.78	.278	.127	.151	.000	.151	77%
1½-Inch Maximum												
70%	1—5	1.86	6.05	3.47	31.4	6.66	.481	.108	.154	.219	.219	112%
75%	1—5	2.22	5.73	3.30	37.5	6.63	.436	.106	.141	.189	.189	97%
75%	1—6	2.66	5.73	3.30	37.5	5.66	.447	.108	.144	.193	.193	99%
80%	1—6	3.11	5.40	3.14	43.7	5.69	.402	.105	.132	.165	.165	85%
75%	1—7	3.11	5.73	3.30	37.5	5.01	.458	.111	.148	.198	.198	102%
80%	1—7	3.62	5.40	3.14	43.7	5.01	.413	.108	.135	.169	.169	87%
80%	1—8	4.14	5.40	3.14	43.7	4.42	.419	.110	.137	.172	.172	88%
85%	1—8	4.72	5.05	2.97	49.8	4.32	.366	.103	.121	.142	.142	73%
¾-Inch Maximum: Gap (28—14)												
78%	1—6	3.55	4.91	3.26	50.0	5.57	.352	.154	.198	.000	.198	102%
¾-Inch Maximum: (4—⅜"), (14—8), (48—28), (0—100)												
83%	1—6	4.25	4.90	3.49	59.9	5.57	.272	.000	.272	.000	.272	82%

*The column marked "ratio" is the ratio of the relative density (under the 12th column) to 0.195 for continuous grading for the dominant group and to 0.333 for the case of the last mix with a gap in (No. 4—⅜).

BY W. H. PRICE*

The fineness modulus method of measuring aggregate gradings was first introduced in Bulletin 1 of the Lewis Institute by D. A. Abrams in 1918, and Fig. 3 of this bulletin shows curves that are

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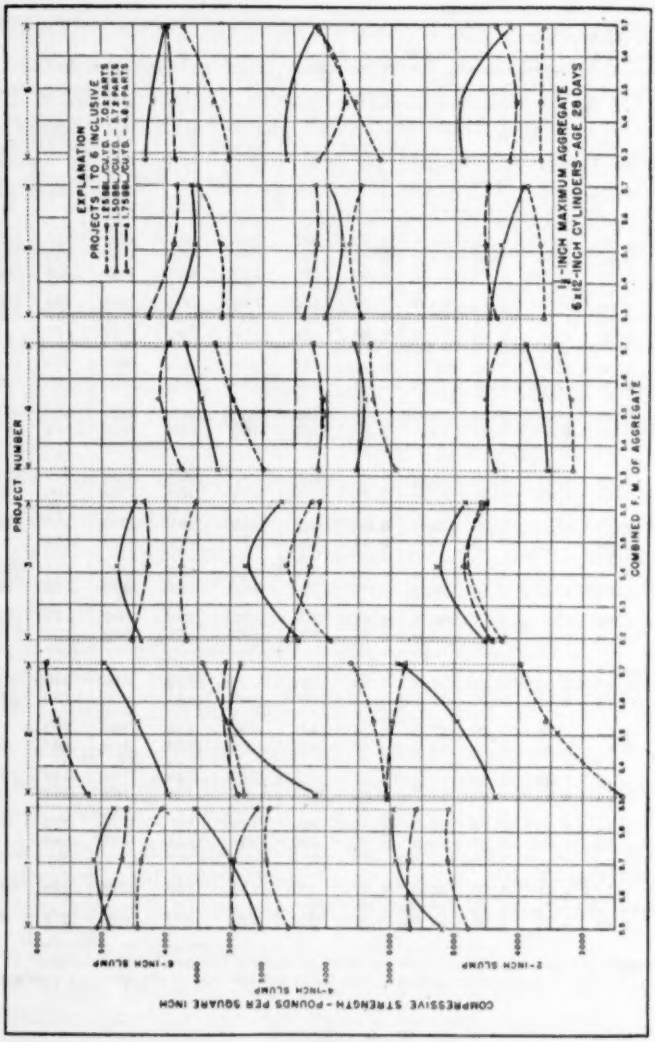

FIG. 9—RELATION BETWEEN FINENESS MODULUS OF AGGREGATE AND STRENGTH OF CONCRETE

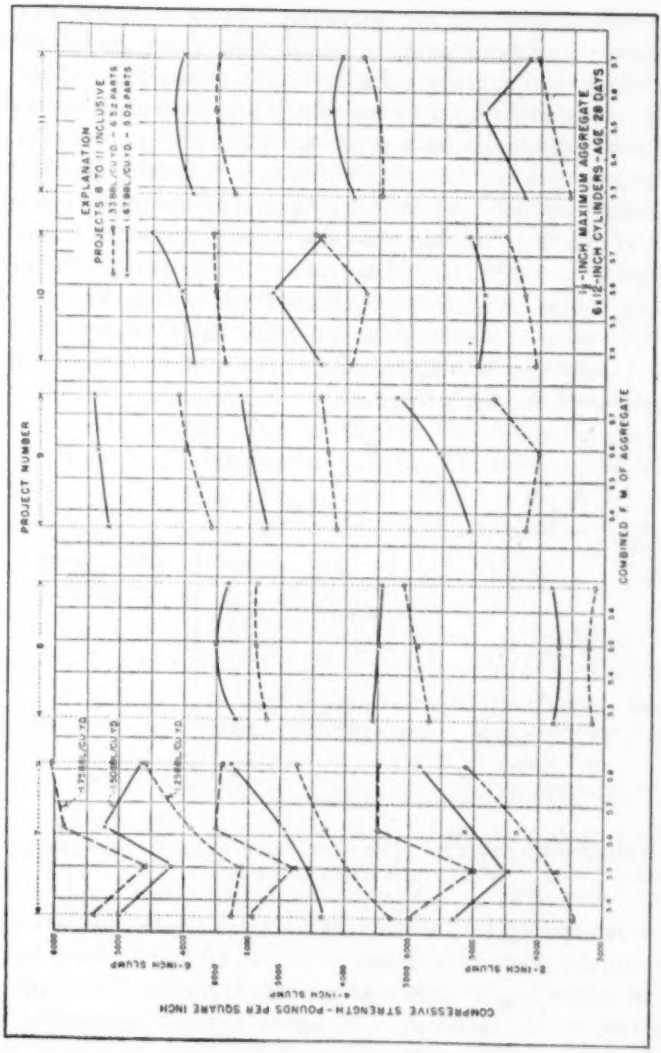

FIG. 10—RELATION BETWEEN FINENESS MODULUS OF AGGREGATE AND STRENGTH OF CONCRETE

very similar in shape to those shown in Fig. 2 of Mr. Kennedy's paper. However, Mr. Abrams concluded from his tests that the variations in strength due to changes in fineness modulus were simply a reflection of the changes in water-cement ratio necessary to produce a given plastic condition; whereas, Mr. Kennedy shows as much as 1,500 p.s.i. difference in compressive strength that may be attributed entirely to changes in aggregate grading. Mr. Kennedy used stepped gradings in the majority of his tests which may account for the differences shown; although, from supplemental tests made with gap gradings he concludes that the compressive strength should vary with the fineness modulus as shown in Fig. 2 of his paper regardless of the type of grading. If this were true then the generally accepted "water-cement ratio law" should be revised to state that, for plastic mixes, using sound and clean aggregate, the strength and other desirable properties of concrete under job conditions are governed by the net quantity of mixing water used per unit of cement *and the fineness modulus of the aggregate.*

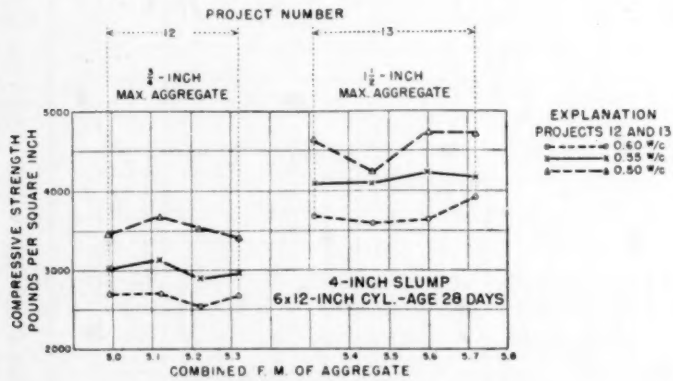

FIG. 11—RELATION BETWEEN FINENESS MODULUS OF AGGREGATE AND STRENGTH OF CONCRETE

In order to investigate the fineness modulus-strength relationship, the writer plotted the curves shown in Fig. 9, 10 and 11 from the test results of 13 sets of mixes chosen at random from the laboratory files of the Bureau of Reclamation. Different natural aggregates from widely separated sources were used in each of the 13 sets of mixes shown. An examination of Fig. 11 shows practically no variation in strength for changes in fineness modulus for mixes having constant water-cement ratio. Fig. 9 and 10 show very little, if any, trend; however, it is known that an average of these results and those of

about 30 other projects shows an increase in strength for increased fineness modulus due mainly to reduced water requirements and lower water-cement ratios for the coarser gradings.

On Fig. 9, projects 1, 3, and 6 show lower strengths on the average for the 1.75 bbl. of cement per cu. yd. mixes than for the 1.50 bbl. of cement per cu. yd. mixes, which is contrary to the generally accepted fact that the strength of concrete for a given set of materials and consistency increases with increased cement content. It appears that Mr. Kennedy's tests are in the same category with those of projects 1, 3, and 6 in that his results are contrary to the generally accepted concept that the grading of the aggregate has but little effect on the compressive strength of concrete within the range of plastic mixes.

In the alternate procedure of mix design (Case 2) given in Mr. Kennedy's paper, it should be mentioned that for a constant cement-content, the mix parts will change slightly in going from a fine to a coarse fineness modulus due to changes in water requirements necessary for maintaining a selected slump and that the compressive strength-fineness modulus curve will differ from that of Case 1 in this respect.

AUTHOR'S CLOSURE

The author can not agree with Mr. Price when he states that the water cement ratio law should be revised to include the effect of variations of fineness modulus on aggregate. A study of the data in the paper (Fig. 4) shows that the fineness modulus has no effect whatever upon strength until the optimum is exceeded. Beyond the optimum, mixes may not be plastic even though they appear to be so when the consistency is measured by the generally accepted slump cone such as was the practice in collecting data for the author's paper.

Without complete data as to type of aggregate, water ratios, etc., it is difficult to analyze Fig. 9, 10 and 11 included in Mr. Price's discussion. His curves are plotted from only three points which apparently are taken near the optimum fineness modulus (See Fig. 2 of paper) and since they cover a range of only three to four tenths in fineness modulus, it could not be expected that the optimum would be clearly discernable. All curves plotted on Fig. 2 of the paper are drawn through at least five points and cover a range in fineness modulus of at least twelve tenths.

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Comparison of the Physical and Mechanical Properties of Hand Rodded and Vibrated Concrete Made With Different Cements*

By George W. Washa†

SYNOPSIS

This paper presents results of a comprehensive series of tests on vibrated and hand-rodded concrete. The variables for each method of placement are five different mixes and five different cements. Particular attention has been paid to the lean mixes, the leanest vibrated mix being 1:6:12, by weight, with only 2.2 sacks of cement per cubic yard of concrete. The results obtained indicate the effects of the variables on compressive strength, permeability, specific weight, absorption, linear changes due to alternate heating and water soaking, modulus of elasticity, Poisson's ratio, shrinkage, and plastic flow.

INTRODUCTION

This research project was begun four years ago in the Materials Testing Laboratory at the University of Wisconsin to obtain a comparison of the properties of vibrated and hand placed concrete made with different cements, and to determine what results might be expected with mixes much leaner than any that are being used for present construction. Three series of tests were made with these objectives in mind.

In Series A, the effects of type of cement, method of placement, and different mix proportions, on the compressive strength, permeability, and density of concrete, were investigated. For these tests, five different cements were used and for each cement 5 vibrated and 5 hand-rodded mixes of different proportions were made. Two hundred compression cylinders, 6 by 12 in., and 200 permeability specimens 9 in. in diameter and 6 in. thick were made.

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The tests of Series B were to determine the effect of type of cement, method of placement, and different mix proportions, on the absorption and linear changes of concrete subjected to soaking, and to cycles of alternate heating and soaking in water. For this series of tests 150 cylinders 6 in. in diameter and 10 in. high were made with the same cements and mix proportions used in Series A.

The object of the tests of Series C was the determination of the effect of type of cement, method of placement, and different mix proportions, on the elastic properties of concrete, as measured by the modulus of elasticity and Poisson's ratio. The effects of these factors on plastic flow of concrete under a constant sustained compressive load of a magnitude equal to that which it might have to carry in an actual structure were also obtained. For these tests 90, 6 by 18 in. cylinders were made. Three vibrated and two hand-rodded mixes made with three different cements were the included variables.

This research project was made possible through the research fund of the Mechanics Department of the University of Wisconsin and with the assistance of N. Y. A. student help. The author is grateful for the opportunity to carry out this program, and is deeply appreciative of the many valuable suggestions given him throughout the course of this work by Prof. M. O. Withey. The author wishes to thank Moulton Basford, Clarence Friday, Gustave Montemayor, Earl Senkbeil, Edwin Saxer, Phillip Dent, and others who assisted in the tests; the Santa Cruz Portland Cement Co., and the Marquette Cement Manufacturing Co. for their cement donations; Prof. K. F. Wendt for his help in securing necessary apparatus; the Forest Products Laboatory for the loan of equipment; and the Chicago, Milwaukee, St. Paul and Pacific Railroad for its loan of the springs used in the plastic flow tests.

MATERIALS

Sand—Janesville Sand containing about 60 per cent quartz and 30 per cent dolomite was used in making all test specimens. Its dry rodded specific weight was 112 lb. per cu. ft., specific gravity, 2.70, voids, 33 per cent, fineness modulus, 2.67. Approximately 99 per cent passed a ¼ in. sieve, 56 per cent passed a No. 30, and 1 per cent passed a No. 100 sieve. Its absorption was 0.6 per cent, by weight.

Gravel—Janesville gravel consisting largely of dolomitic particles was used in making all test specimens. The gravel was screened and graded by weight as follows: 1 part ¼ to ⅜ in., 2 parts ⅜ to ¾ in., and 2 parts ¾ to 1½ in. in diameter. The fineness modulus of this combined aggregate was 7.10, rodded weight per cu. ft. 105 lb., specific gravity, 2.68, and absorption, 1.10 per cent, by weight.

Cements—The five different cements used consisted of: one standard portland cement, designated 1; one low heat cement, 3; two high early strength cements, 2 and 5; and one high silica cement (designated 4) made by grinding together 70 per cent portland cement clinker and 30 per cent high silica compound of puzzolanic origin. All cements were stored in a large galvanized iron bin made as nearly air tight as possible. The chemical analyses, as furnished by the manufacturers, are given in Table 1, the compounds present in Table 2, and the physical properties as determined in the University of Wisconsin testing laboratory, Table 3.

TABLE 1—CHEMICAL ANALYSIS OF CEMENT—MANUFACTURER'S DATA

Cement	SiO_2	Fe_2O_3	Al_2O_3	C_aO	MgO	SO_3	Loss on Ignition	Insoluble Residue
1	20.32	2.78	5.66	63.68	4.17	1.73	0.88	....
2	19.64	3.02	6.48	66.02	1.07	2.25	1.21	....
3	21.8	5.8	5.0	59.8	4.0	1.6	0.8	0.1
4	33.80	2.27	6.83	52.75	2.02	1.19	2.00	....
5	20.42	3.46	6.48	62.97	2.52	2.37	1.28	0.33

TABLE 2—CALCULATED COMPOUNDS

Cement	C_3S	C_2S	C_3A	C_4AF	$CaSO_4$
1	57.8	14.7	10.3	8.5	2.9
2	65.0	7.4	12.1	9.2	3.8
3	31.4	38.8	3.4	17.6	2.7
4*	53.1	24.6	12.4	7.1	....
5	45.7	24.1	11.3	10.5	4.0

*Compounds present in the clinker. The cement consisted of 70 per cent portland cement clinker ground with 30 per cent high silica compound.

MAKING SPECIMENS

Vibrators—An internal vibrator with a spud 1¾ in. in diameter, 20 in. long, weighing 7.7 lb. was used. This spud was attached by a flexible shaft to a universal ½ h.p. motor designed to run at 4500 r.p.m. on a 110 volt D.C. or single phase A.C. circuit. The actual measured speed of vibration of the spud in the concrete was about 3300 r.p.m.

Proportions—The mix proportions used in this work were determined by the water-cement ratio trial method of proportioning concrete mixtures. The vibrated mixes were designed for zero slump while the hand-rodded mixes were designed for approximately a 2-in. slump. Table 4 summarizes the values of the water-cement ratios and the mix proportions used.

Mixing of Concrete—Air dry aggregates were used in all cases, and allowance was made for the water absorbed by the aggregates. All mixing was done thoroughly by hand in metal mixing trays, with the

TABLE 3—PHYSICAL TESTS OF CEMENT

Property	Used in Series A and B					Used in Series C		
	Cement							
	1	2	3	4	5	1	2	4
Normal Consistency	23.5	25.0	22.0	31.0	27.0	23.5	25.0	31.0
Initial Set	3 h 0 m	2 h 45 m	3 h 0 m	2 h 30 m	2 h 0 m	3 h 30 m	2 h 30 m	2 h 0 m
Final Set	5 h 30 m	4 h 30 m	5 h 30 m	4 h 45 m	3 h 30 m	5 h 30 m	4 h 0 m	4 h 30 m
Soundness	OK	OK	OK	OK	OK	OK	OK	OK
Fineness No. 200 Sieve	93.7	99.3	95.6	95.2	98.7	91.8	99.0	99.0
Fineness No. 325 Sieve	85.1	94.1	89*	93.6	98.7	....	....	....
Surface Area	1455	1970	1850*	2075	2770	....	....	....
Specific Gravity	3.08	3.12	3.16	2.87	3.12	....	....	....
Tensile Strength of 1:3 Standard Mortar Briquettes								
1 day	135	358	142	177	345	160	330	195
3 day	260	437	258	315	425	245	380	305
7 day	340	473	299	391	455	320	460	405
28 day	400	490	398	455	500	400	450	490
60 day	395	473	460	515	465	370	440	485
180 day	380	472	490	540	475	375	460	520
360 day	305	455	460	540	430	370	470	525
720 day	330	440	410	525	440	355	430	545
Compressive Strength of 1:2.75 Mortar 2 in. Cubes								
1 day	502	....	....	....	1405	....	....	....
3 day	1398	3920	1032	2451	4520	....	....	....
7 day	2587	4716	1525	3800	5505	3145	5000	4125
28 day	4179	5797	2925	5775	6805	4820	6155	6700
60 day	3932	5832	4000	6018	7625	....		
180 day	4188	6097	5210	6343	7330	....		
360 day	4370	5990	5000	6697	7000	5745	6615	7365
720 day	4405	6105	4925	6565	6905	5160	6210	7400

*Furnished by manufacturer.

TABLE 4—MIX PROPORTIONS

$\frac{W}{C}$ Ratio, by Weight	Vibrated Mixes, by Weight, 0 Slump		Hand-Rodded Mixes, by Weight, 2-in. Slump	
	Mix	Proportions	Mix	Proportions
.491	A	1:3:7	E	1:2:4½
.584	B	1:4:8	F	1:3:5
.660	C	1:4¼:9½	G	1:3½:6
.750	D	1:5:10	H	1:4:7
.850	J	1:6:12	K	1:5¼:8½

batches varying in weight but in all cases under 200 lb. After mixing, the batches were allowed to stand under damp canvas for approximately 10 minutes, then remixed and placed in the molds.

Placing of Concrete—The 2-in. slump concrete was placed in the molds in the usual manner. The 0-slump concrete was placed in molds provided with false tops, the spud was pushed to within an inch of the bottom and moved up and down until a water film appeared on the top surface. The time of vibration was variable but in practically all cases was less than one minute.

Curing—The concrete was kept in the molds under damp canvas for 48 hours after which the molds were removed and the concrete then

placed in the moist closet until it was 28 days old. In Series A and C the concrete was tested at the age of 28 days but in Series B the concrete was taken out of the moist closet after this period, and then stored in the laboratory for approximately another 30 days before beginning the test cycles.

TESTING

Tests of Series A—The compressive strength tests were run in accordance with standard procedure.

The permeability tests were performed on 9½ in. by 6 in. cylinders after moist curing for 28 days. The procedure and apparatus used in conducting these tests have been described by the author[1]. The criterion used to measure permeability for this series of tests was the average flow of water into the concrete from the 40th to the 50th hour of test.

Specific weight determinations were made on all specimens after removal from the molds by weighing in air and under water. The calculations for cement contents, solidity ratios, and voids in Tables 5, 6, 7, and 8, were based on these specific weights and the actual quantities present in the freshly mixed concrete.

Tests of Series B—The specimens were provided with top and bottom brass plugs which projected out of the concrete about ⅝ in. The expansions due to the alternate heating and soaking cycles were read with a 1/10,000th Federal dial in an assembly similar to that used in autoclave test measurements. Each cycle consisted of 9 hours of heating to a maximum temperature of 180°F., 48 hours of water storage, and 15 hours of air storage.

Tests of Series C—In these tests, six cylinders were made for each variable condition. Two were tested for modulus of elasticity values and were then failed to determine the 28-day compressive strength. Two were tested for modulus of elasticity and Poisson's ratio values to one-third of the ultimate load given by the first two, and were later used as control specimens in the plastic flow tests. Longitudinal strains were measured with dial compressometers placed on diametrically opposite sides of the specimen, and the lateral strains were measured with a Lamb's Lateral Extensometer. The last two specimens were placed in the plastic flow assemblies, similar to those previously reported, and were stressed to one-fourth of their ultimate strengths or to the maximum spring force. The specimens in the plastic flow assemblies and the control specimens were placed in the same room in which the temperature varied from 60 to 80°F., and the relative humidity from 25 to 60 per cent.

[1]All references are listed at the end of this paper.

RESULTS OF TESTS—SERIES A

General Considerations—The important results obtained in this group of tests are presented in Tables 5 to 8, and Fig. 1 to 8. The curves are marked according to type of cement and method of placement used. For example: 3 V refers to the vibrated mixes of cement 3, while 3 refers to the hand-rodded mixes of the same cement.

The permeability information should be regarded as qualitative rather than quantitative because the values reported from the small test specimens are probably not directly comparable to conditions in an actual structure, also it is quite probable that values reported by different laboratories would vary materially because of differences in apparatus and test procedure.

The test data clearly show that the solidity ratios are higher for the vibrated mixes in all cases. It is also evident that the solidity ratios are definitely higher for compression specimens than for the permeability specimens for the vibrated mixes, but that there is no appreciable difference in these values for the hand-rodded specimens. This observation points to the conclusion that the efficiency of vibration in laboratory use is dependent to some extent on the form and dimensions of the mold.

Effect of Water-Cement Ratio and Type of Cement on Strength—The data in Fig. 1 show that the usual relation between water-cement ratio and strength holds for the hand-rodded and the vibrated mixes, and also show that for a given water-cement ratio the vibrated mixes in all instances had higher strengths. The differences between the strength values of vibrated and hand-rodded concretes for a given water-cement ratio and cement varied from 125 to 1065 p.s.i. The effect of the type of cement on the strength results is also clearly brought out. The two high early strength cements, 2 and 5, and the high silica cement 4, gave the highest results, the standard cement 1 gave lower values, while the low heat cement 3, gave the lowest values. Since the strength tests were made at 28 days these results point out larger differences in cement behavior than would probably be obtained at later ages. It is to be noted that for the leanest vibrated mix, 1:6:12, strengths of approximately 2500 p.s.i. were obtained for cements 2, 4, and 5. The hand-rodded strengths for the same w/c ratio and same cements varied from 1900 to 2330 p.s.i.

Effect of Water-cement Ratio and Type of Cement on Permeability—The data of Fig. 2 show that as the water-cement ratio increased the inflow of water increased for both the vibrated and hand-rodded mixes, also that for a given water-cement ratio the vibrated concrete

TABLE 5—PROPERTIES OF VIBRATED MIXES USED IN STRENGTH TESTS

Proportions, by Weight	Water-Cement Ratio, by Weight w/c	Cement	Specific Weight, Lb./Ft.³	Cement Content: Sacks per Cu. Yd.	Cement Content: By Absolute Vol./c	Cement Void Ratio c/v	Cement Water Ratio c/w	Air Voids	Solidity Ratio p	28 Day Compressive Strengths p.s.i.
		1	156.1	3.870	.0701	.5580	.6610	.0195	.8744	4730
		2	156.0	3.870	.0691	.5445	.6520	.0209	.8731	6310
1:3:7	.491	3	155.8	3.865	.0682	.5300	.6440	.0227	.8714	3490
		4	154.1	3.823	.0744	.5655	.7104	.0268	.8685	6120
		5	156.3	3.875	.0693	.5623	.6525	.0170	.8768	6275
		1	156.0	3.273	.0593	.4740	.5560	.0185	.8749	3970
		2	155.6	3.262	.0583	.4530	.5485	.0224	.8713	5200
1:4:8	.584	3	155.0	3.251	.0574	.4345	.5425	.0263	.8679	2530
		4	154.1	3.233	.0629	.4780	.5975	.0264	.8684	5220
		5	155.1	3.255	.0582	.4433	.5500	.0255	.8687	5410
		1	154.5	2.860	.0518	.3900	.4922	.0277	.8671	2830
		2	155.2	2.880	.0515	.3970	.4872	.0240	.8703	4530
1:4¼:9½	.660	3	155.1	2.865	.0506	.3855	.4790	.0256	.8688	2130
		4	154.0	2.842	.0553	.4175	.5273	.0275	.8676	4295
		5	155.3	2.867	.0513	.3967	.4855	.0236	.8707	4405
		1	154.4	2.628	.0476	.3540	.4325	.0247	.8653	2565
		2	154.7	2.633	.0470	.3515	.4265	.0236	.8663	3490
1:5:10	.750	3	153.9	2.616	.0462	.3340	.4220	.0290	.8615	1420
		4	153.1	2.603	.0506	.3655	.4645	.0296	.8614	3430
		5	154.6	2.625	.0470	.3500	.4267	.0242	.8657	3715
		1	153.6	2.200	.0399	.2918	.3820	.0324	.8632	1515
		2	153.8	2.210	.0394	.2882	.3770	.0322	.8633	2470
1:6:12	.850	3	152.3	2.185	.0386	.2668	.3724	.0409	.8555	1120
		4	153.0	2.192	.0426	.3112	.4097	.0330	.8630	2450
		5	154.5	2.215	.0396	.3002	.3770	.0268	.8681	2695

TABLE 6—PROPERTIES OF HAND-RODDED MIXES USED IN STRENGTH TESTS

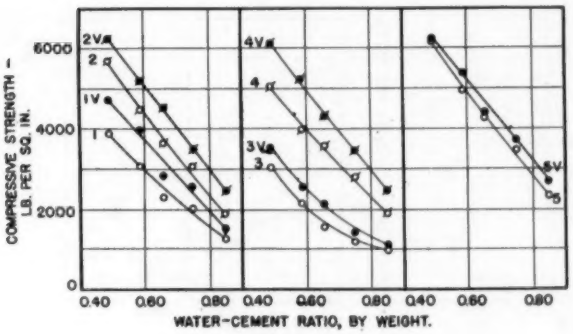
Proportions, by Weight	Water-Cement Ratio, by Weight w/c	Cement	Specific Weight, Lb./Ft.³	Cement Content: Sacks per Cu. Yd.	Cement Content: By Absolute Vol./c	Cement Void Ratio c/v	Cement Water Ratio c/w	Air Voids	Solidity Ratio p	28 Day Compressive Strengths p.s.i.
		1	153.6	5.480	.0993	.6155	.6620	.0114	.8386	3905
		2	153.5	5.470	.0980	.5995	.6540	.0136	.8366	5700
1:2:4½	.491	3	153.6	5.480	.0968	.5910	.6460	.0137	.8363	3020
		4	151.3	5.395	.1049	.6293	.7090	.0188	.8333	5055
		5	152.3	5.435	.0972	.5720	.6535	.0213	.8300	6150
		1	152.9	4.550	.0824	.5065	.5570	.0146	.8374	3080
		2	152.5	4.533	.0811	.4895	.5490	.0179	.8344	4495
1:3:5	.584	3	152.3	4.530	.0800	.4780	.5425	.0199	.8326	2170
		4	150.6	4.476	.0870	.5155	.5962	.0228	.8313	3940
		5	152.2	4.530	.0810	.4895	.5490	.0180	.8345	4950
		1	152.4	3.890	.0705	.4353	.4920	.0187	.8380	2295
		2	152.1	3.880	.0694	.4225	.4850	.0212	.8357	3630
1:3½:6	.660	3	151.9	3.880	.0685	.4120	.4800	.0231	.8341	1585
		4	151.1	3.859	.0750	.4573	.5275	.0219	.8360	3575
		5	151.8	3.870	.0693	.4173	.4855	.0234	.8339	4250
		1	151.6	3.390	.0614	.3726	.4335	.0231	.8352	2015
		2	151.4	3.380	.0604	.3605	.4270	.0260	.8325	3055
1:4:7	.750	3	151.1	3.370	.0595	.3515	.4210	.0280	.8306	1200
		4	150.9	3.370	.0655	.3974	.4643	.0238	.8352	2800
		5	152.0	3.400	.0608	.3715	.4280	.0215	.8364	3465
		1	150.4	2.743	.0497	.2978	.3816	.0367	.8330	1250
		2	150.4	2.746	.0490	.2925	.3763	.0375	.8323	1915
1:5¼:8½	.850	3	150.3	2.740	.0484	.2868	.3713	.0385	.8314	990
		4	149.5	2.727	.0530	.3146	.4095	.0389	.8316	1900
		5	151.7	2.765	.0496	.3085	.3770	.0292	.8395	2330

TABLE 7—PROPERTIES OF VIBRATED MIXES USED IN PERMEABILITY TESTS

Proportions, by Weight	Water-Cement Ratio, by Weight w/c	Cement	Specific Weight Lb./Ft.³	Cement Content: Sacks per Cu. Yd.	Cement Content: By Absolute Vol.	Cement Void Ratio c/v	Cement Water Ratio c/w	Air Voids	Solidity Ratio p	40-50 Hr. Inflow, Millionths Gal. per Sq. Ft. per Hr.
		1	155.3	3.854	.0698	.5355	.6620	.0247	.8698	152
		2	155.8	3.870	.0691	.5395	.6520	.0222	.8719	92
1:3:7	.491	3	155.5	3.858	.0681	.5210	.6450	.0251	.8692	167
		4	153.4	3.803	.0740	.5460	.7100	.0313	.8645	25
		5	156.2	3.870	.0692	.5525	.6525	.0192	.8747	50
		1	154.3	3.240	.0587	.4370	.5570	.0289	.8657	224
		2	154.5	3.240	.0579	.4300	.5490	.0291	.8654	112
1:4:8	.584	3	154.9	3.251	.0574	.4322	.5430	.0271	.8672	325
		4	152.2	3.194	.0621	.4365	.5975	.0383	.8577	50
		5	154.5	3.240	.0580	.4313	.5490	.0289	.8655	75
		1	154.4	2.853	.0517	.3865	.4920	.0287	.8662	617
		2	154.0	2.845	.0508	.3710	.4847	.0321	.8631	150
1:4¼:9½	.660	3	154.5	2.855	.0504	.3754	.4790	.0291	.8657	634
		4	152.6	2.817	.0548	.3905	.5274	.0363	.8598	46
		5	154.6	2.853	.0511	.3824	.4848	.0282	.8664	113
		1	153.5	2.610	.0473	.3393	.4332	.0302	.8606	850
		2	153.6	2.612	.0467	.3340	.4270	.0304	.8602	175
1:5:10	.750	3	154.0	2.616	.0462	.3348	.4215	.0284	.8620	2675
		4	152.4	2.590	.0504	.3538	.4645	.0339	.8576	50
		5	153.1	2.605	.0466	.3273	.4271	.0333	.8576	142
		1	153.0	2.200	.0398	.2840	.3832	.0362	.8598	139700
		2	153.1	2.198	.0392	.2817	.3765	.0351	.8608	2530
1:6:12	.850	3	151.9	2.177	.0384	.2603	.3722	.0444	.8525	29650
		4	152.3	2.185	.0425	.3012	.4102	.0375	.8589	146
		5	153.8	2.200	.0394	.2923	.3765	.0302	.8652	190

TABLE 8—PROPERTIES OF HAND-RODDED MIXES USED IN PERMEABILITY TESTS

Proportions, by Weight	Water-Cement Ratio, by Weight w/c	Cement	Specific Weight Lb./Ft.³	Cement Content: Sacks per Cu. Yd.	Cement Content: By Absolute Vol.	Cement Void Ratio c/v	Cement Water Ratio c/w	Air Voids	Solidity Ratio p	40-50 Hr. Inflow, Millionths Gal. per Sq. Ft. per Hr.
		1	154.3	5.500	.0997	.6335	.6615	.0068	.8426	117
		2	153.4	5.470	.0980	.5975	.6545	.0143	.8360	67
1:2:4½	.491	3	153.2	5.470	.0966	.5830	.6455	.0160	.8344	125
		4	152.2	5.433	.1056	.6525	.7100	.0132	.8382	100
		5	152.7	5.450	.0974	.5810	.6535	.0186	.8324	75
		1	153.2	4.560	.0826	.5160	.5570	.0119	.8398	275
		2	152.6	4.533	.0811	.4920	.5490	.0171	.8351	75
1:3:5	.584	3	152.6	4.540	.0802	.4850	.5425	.0175	.8346	200
		4	150.8	4.480	.0871	.5195	.5956	.0215	.8323	100
		5	152.1	4.525	.0809	.4870	.5490	.0187	.8339	109
		1	152.8	3.910	.0708	.4444	.4928	.0156	.8407	1110
		2	152.3	3.885	.0695	.4260	.4853	.0200	.8368	192
1:3½:6	.660	3	151.8	3.880	.0685	.4109	.4800	.0241	.8332	467
		4	151.0	3.859	.0750	.4555	.5277	.0227	.8353	92
		5	152.1	3.880	.0694	.4247	.4853	.0204	.8366	157
		1	152.2	3.400	.0616	.3810	.4330	.0196	.8382	2832
		2	151.9	3.392	.0606	.3673	.4267	.0230	.8350	225
1:4:7	.750	3	151.6	3.390	.0599	.3602	.4223	.0245	.8337	1575
		4	151.2	3.380	.0657	.4030	.4645	.0216	.8370	133
		5	152.1	3.400	.0608	.3725	.4280	.0210	.8368	224
		1	149.6	2.735	.0495	.2882	.3819	.0420	.8283	171400
		2	150.7	2.748	.0491	.2947	.3763	.0361	.8334	20075
1:5¼:8½	.850	3	150.3	2.740	.0484	.2868	.3713	.0385	.8314	45550
		4	149.3	2.722	.0529	.3120	.4092	.0403	.8304	347
		5	151.1	2.755	.0493	.3012	.3770	.0329	.8363	538



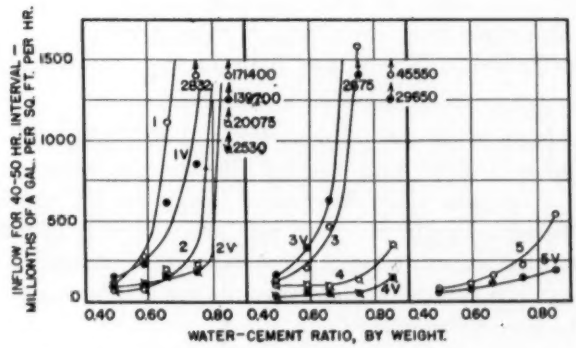


FIG. 1—RELATION OF STRENGTH AND WATER-CEMENT RATIO
FIG. 2—RELATION OF PERMEABILITY AND WATER-CEMENT RATIO

in general was less permeable. With the two lowest water-cement ratios for both vibrated and hand-rodded mixes fairly low inflows were obtained with all cements, but for the three highest water-cement ratios large differences were obtained. Mixes made with cements 4 and 5 gave exceptionally good results, while those made with cements 1 and 3 gave poor results with the leaner mixes. It is to be noted that even for mixes as lean as 1:6:12 vibrated or 1:5¼:8½ hand-rodded, satisfactory concrete can be obtained from the standpoint of permeability with certain cements.

Relation Between Water-Cement Ratio and Air Voids—The data in Tables 5, 6, 7, and 8 show that there is a definite tendency for the air voids to increase with an increase in the water-cement ratio. It is seen that the volume of air voids in the vibrated concrete, as a rule is greater than in the hand-rodded mixes of the same water-cement ratio with the exception of the highest one, .850. No conclusions can

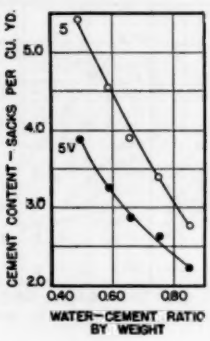


FIG. 3—RELATION OF CEMENT CONTENT AND WATER-CEMENT RATIO

be drawn as to the effect of type of cement on the volume of air voids. The data also show that there was a greater volume of air voids in the vibrated permeability mixes than in the vibrated compression mixes.

Relation Between Water-Cement Ratio and Specific Weight—The data in Tables 5, 6, 7, and 8 show that as the water-cement ratio increased the specific weight decreased, also that the specific weights for the vibrated mixes in all cases were considerably greater than for the hand-rodded mixes for any given water-cement ratio. It should be noted that the type of cement used in the mix seems to have little effect on the specific weight.

Relation Between Water-Cement Ratio and Cement Content—The data in Fig. 3 are plotted for cement 5 only, but the curves for the other cements would practically fall on the same lines. The data show that as the water-cement ratio was increased the cement content was decreased, and also, that for any given water-cement ratio there was much less cement used in the vibrated mixes than in the hand-rodded mixes. It is interesting to note that the cement content for the 1:6:12 vibrated mix was only about 2.2 sacks per cu. yd.

Relation Between Strength, Permeability, and Cement-Voids Ratio—Curves plotted with strengths as ordinates and cement-voids ratios as abscissas would show the usual rise in strength with an increase in the cement-voids ratio, and would also point out the superiority of the vibrated mixes. If inflow of water were plotted against the cement-voids ratios the curves would show decreasing inflow with higher cement-voids ratios. The vibrated mixes would again show better results.

Relation Between Strength and Permeability—The data of Fig. 4 show that the inflow decreased with an increase in strength. It can be con-

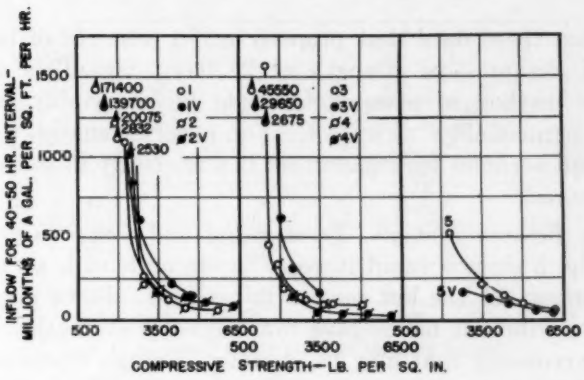
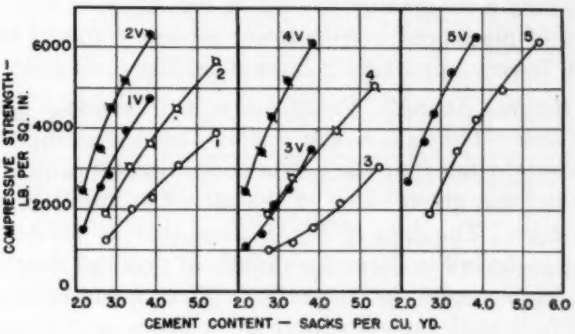


FIG. 4—RELATION OF PERMEABILITY AND STRENGTH



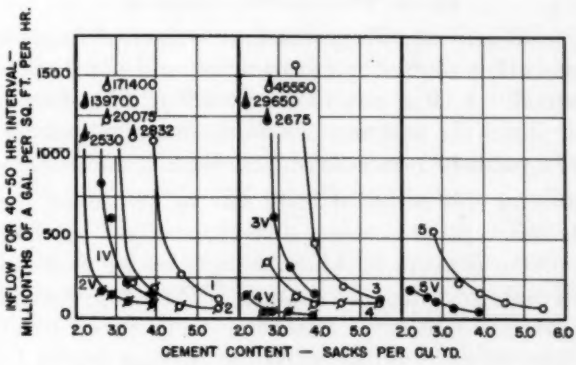


FIG. 5—RELATION OF STRENGTH AND CEMENT CONTENT

FIG. 6—RELATION OF PERMEABILITY AND CEMENT CONTENT

cluded from these data that properly cured concrete of better than 3000 p.s.i. compressive strength at 28 days, regardless of type of cement or method of placement, would be reasonably satisfactory from the permeability viewpoint. For lower strengths the type of cement used becomes very important if a relatively impermeable concrete is desired.

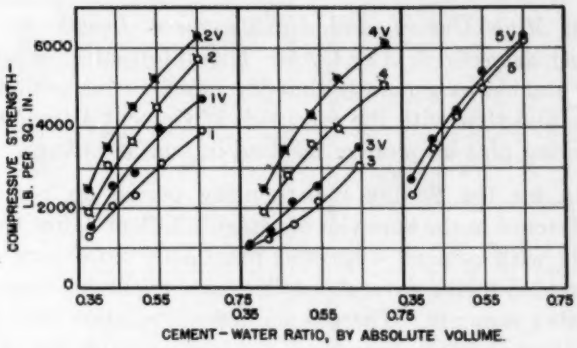
Relation Between Strength, Permeability and Cement Content—The data of Fig. 5 show a rapid increase in strength with an increase in cement content for the low cement mixes used. For a given cement content the vibrated mixes gave much greater strengths, thus indicating the economy that may be obtained through vibration. Fig. 6 shows that the inflow decreased very definitely as the cement content was increased and that for a given cement content the vibrated mixes showed considerably less inflow. It can be said that if properly cured concrete has at least 4½ sacks of cement per cubic yard of concrete it will allow only a reasonably low inflow regardless of type of cement and method of placement. With lower cement contents the type of cement used is very important if impermeability is desired.

Relation Between Strength, Permeability, and Cement-Water Ratio by Absolute Volume—The data of Fig. 7 show that the strength increases, almost as a straight-line function, as the cement-water ratio is increased; and also that for a given value of the ratio the vibrated mixes gave higher strengths. The data of Fig. 8 show that as the cement-water ratio increases, the inflow decreases rapidly at first and then at a slower rate for the higher values of the ratio. Mixes made with cements 1, 2, 4, and 5 indicated generally that for a given cement-water ratio, vibration produced more impermeable concrete.

RESULTS OF TESTS—SERIES B

General Considerations—The problem of volume changes in concrete due to causes other than stress is complex and one that is not now fully understood. A large amount of work has been done by investigators on air shrinkage and expansion due to water soaking, but very little has been published on a combined heating and soaking cycle.

The purpose of this series of tests was to determine the relations existing between type of cement, mix proportions, method of placement and volume constancy, when the specimens were subjected to cycles of alternate heating and soaking. The cycle for this purpose was chosen to simulate in a somewhat exaggerated manner actual field conditions, namely, an intensive hot drying period followed by sudden water cooling. The maximum temperature to which specimens were heated, 180°F., was undoubetdly higher than found in the field



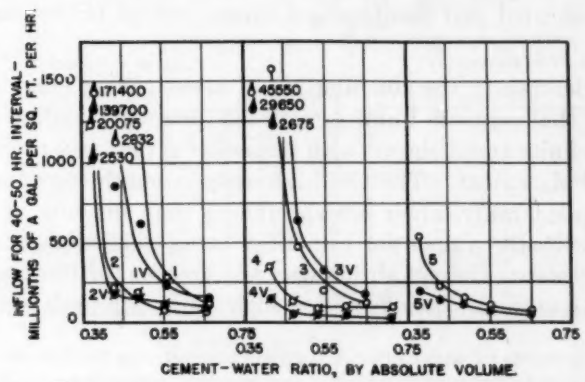


FIG. 7—RELATION OF STRENGTH TO CEMENT WATER RATIO
FIG. 8—RELATION OF PERMEABILITY TO CEMENT-WATER RATIO

but it was felt that the use of the higher temperature would only hasten the reactions somewhat without qualitatively changing the results. The visual changes that occurred during the test period were the formation of a thin white layer, probably $Ca(OH)_2$, on the top surface of the specimens, and a tendency for parts to chip and break off rather easily. The concrete specimens with a high water-cement ratio showed greater disintegration effects.

Examination of the data in this series with respect to the effect of compound composition showed no relation between volume changes and composition. F. M. Lea and C. H. Desch[2] in commenting on shrinkage of portland cement remark that since the results of recent investigations are very conflicting, there are no possible conclusions to be drawn regarding the relative effects of the various compounds on shrinkage. The various cements used showed different behaviors in these tests, as is indicated later, but there seemed to be no definite relation to composition.

Effects of Moist Curing, and Air Storage on Lengths of Specimens Before Being Subjected to Any Cycle—This information is summarized in Table 9 and shows generally that the moist curing period produced expansions, and that with the exception of Cement 4 the combination of moist curing plus air storage resulted in final shrinkage.

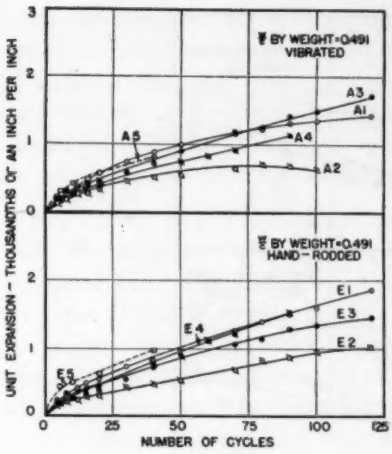
The data for the 26-day moist curing period do not show any definite difference in the behavior of cements 1, 2, or 3, but the vibrated mixes made with cement 4 showed practically no expansions, while the hand-rodded mixes gave about the same results as those obtained with the other cements. There is no definite relation shown between mix proportions and the expansions, nor is there any marked difference between vibrated and hand-rodded mixes, except for cement 4 (high silica).

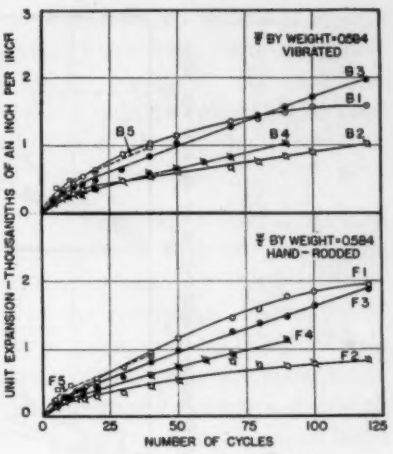
With reference to the combination of moist curing plus dry storage it is noted that cements 1 and 3 exhibited somewhat similar shrinkages with no definite trend shown with respect to either mix proportions or method of placement. The two high early strength cements, 2 and 5, showed considerably larger net shrinkages than did any of the other cements, with the values for cement 5 being particularly large, also, for these cements the net shrinkages was greater for the hand-rodded mixes. Greater air shrinkage for high early strength cements has

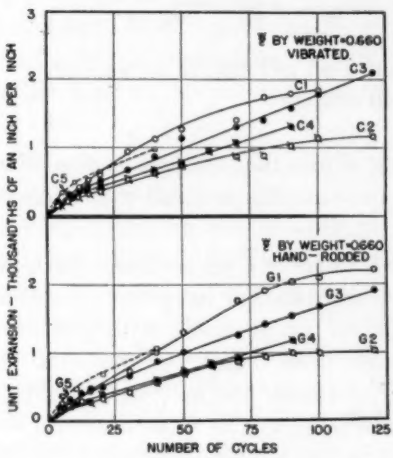
TABLE 9—EFFECTS OF MOIST CURING, AND AIR STORAGE ON LENGTH OF SPECIMEN

Mix	Proportions, by Weight	Water-Cement Ratio, by Weight	Method of Placement	Change per Unit Length Due to 26 Days Moist Curing Millionths of Ins. per In.					Change per Unit Length Due to 26 Days Moist Curing Followed by 30 Days Dry Storage, Millionths of Inches per In.				
				Cement					Cement				
				1	2	3	4	5	1	2	3	4	5
A	1:3:7	0.491	Vib.	28	37	12	0	—	−18	−27	−47	13	−93
B	1:4:8	0.584	Vib.	45	30	15	0	—	−22	−58	−25	28	−155
C	1:4¼:9½	0.660	Vib.	30	58	7	7	—	−28	−47	−25	30	−132
D	1:5:10	0.750	Vib.	30	13	22	0	—	−32	−92	−17	32	−125
J	1:6:12	0.850	Vib.	30	13	35	2	—	−13	−62	−15	27	−103
E	1:2:4½	0.491	Hand	33	28	18	45	—	−23	−85	−68	18	−185
F	1:3:5	0.584	Hand	32	38	22	36	—	−23	−87	−20	15	−183
G	1:3½:6	0.660	Hand	27	15	20	40	—	−48	−105	−13	37	−193
H	1:4:7	0.750	Hand	28	28	47	33	—	−25	−115	−47	23	−192
K	1:5¼:8½	0.850	Hand	8	25	10	37	—	−20	−73	−18	27	−165

also been reported by W. H. Glanville[3], Inge Lyse[4], and other investigators. While cements 1, 2, 3, and 5 showed that the 30 day dry storage period following the initial 26 days moist curing resulted in net shrinkages, this was not true for cement 4. The vibrated mixes made with this cement showed actual expansions due to the air drying, while the hand-rodded mixes showed slight decreases in the expansions resulting from moist curing.







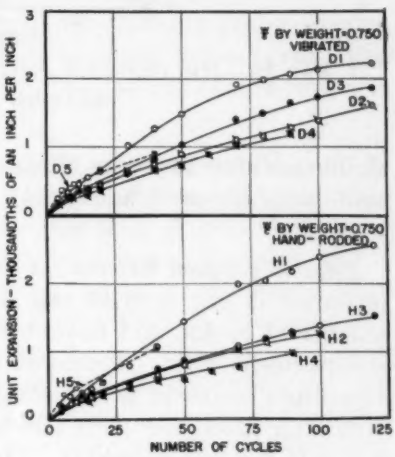


FIG. 9 FIG. 10

FIG. 11 FIG. 12

The data obtained in connection with the plastic flow tests, Table 12, show shrinkages due to 28 days exposure to laboratory air, after the usual 28 days of moist curing, for all mixes used including those made with cement 4. The 28-day shrinkages in Table 12 are larger in all cases than the net shrinkages from Table 9, but neither the storage conditions nor the cements used were exactly the same, and also the shrinkage measurements in Table 12 were made on specimens that had been previously loaded, for a short time, to one-third of their ultimate strengths after 28 days moist curing. Examination of the

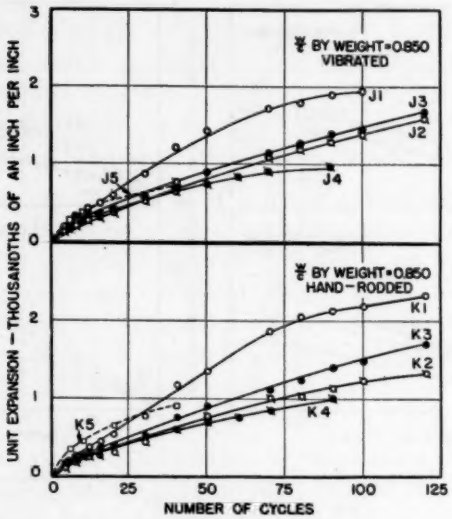


FIG. 13—RELATION OF EXPANSION TO CYCLES OF ALTERNATE HEATING AND SOAKING

shrinkages after two years (Table 12) shows that ultimately the mixes made with cement 4 had shrunk more than those made with cements 1 and 2.

Relation Between Expansions and Number of Cycles—These data are presented in Fig. 9 to 13 and show the relation between expansion, number of cycles, and cement type for mixes made with the same water-cement ratio. Examination of these figures shows that the expansions increased as the number of cycles was increased and that even after 120 cycles there still was a tendency in most cases for the curves to rise rather sharply. There is also a definite trend indicating that the high water-cement ratio mixes suffered the largest expansions, but there seems to be no difference in the performances of the vibrated mixes as against the hand-rodded mixes. Concrete made with cements 2 and 4 had the lowest expansions, while that made with cement 1 usually had the highest. Mixes made with cement 2 had the lowest expansions for the three lowest water-cement ratios, while those made with cement 4 had the lowest expansions for the two highest water-cement ratios. The largest expansion obtained was 2555 millionths of an inch per inch which occurred at 120 cycles for mix H made with cement 1.

It is generally agreed that a cycle of drying and soaking will result in an increase in volume and that this effect is due to continued hydra-

tion of the cement. R. W. Carlson[5] reports that examination of concrete specimens three years after placement has shown various degrees of hydration ranging from partial to almost complete. It would be expected that the cycle used in these tests would be favorable to continued hydration and that consequently one of the reasons for expansion would be the continued hydration of the cement.

Also, during hydration of the cement calcium hydroxide is formed which is soluble in water and would be expected to leach out of the concrete. That this action took place was evident by the formation of a layer of this material on the tops of the specimens. Removal of the calcium hydroxide would result in a more porous material and allow easier and faster access of the water to the inner portions of the concrete specimens. Reaction of compounds in the storage water with the free lime in the concrete may have resulted in compounds of large volume with consequent expansions. The magnitude of this effect would have been dependent, in part, upon the porosity of the concrete specimens, and consequently would have produced greater expansions in the high water-cement ratio mixes. This, as has been noted, was the case.

Complete explanation of the expansions obtained would seem to combine the effects already mentioned with one that would be expected to happen during the heating period. Heating of the water-soaked cylinders, up to 180°F., would accelerate chemical reactions and it might be expected that the largest amount of the reactions took place during the heating period. This would mean that the hydration and reaction products would be formed when the concrete was in a highly expanded state due to the high temperature, and that when the concrete was again allowed to come back to room temperature the newly formed compounds would have prevented return to original dimensions. The greater porosity of the mixes with high water-cement ratios would have allowed more chemical action to take place with consequent production of greater expansions.

Relation Between 48 Hour Absorption Values Determined After Completing Certain Cycles, Cement Type, and Mix Proportions—The data presented in Table 10 show that the absorption increased with an increase in the water-cement ratio. In the vibrated group, mix A with a water-cement ratio of 0.491 had the lowest absorptions while the highest occurred with mix J which had a water-cement ratio of 0.85. For the hand-rodded group mix E, water-cement ratio 0.491, had the lowest absorptions, while mix K, water-cement ratio 0.850, had the highest. The data also show that the absorptions decreased

as the specimens were subjected to a greater number of cycles of heating and soaking. Mixes made with the two high early strength cements, 2 and 5, showed relatively low absorptions as did the mixes made with the high silica cement, 4. Mixes which were made with cement 1 showed the greatest absorptions and as previously noted also showed the greatest expansions.

If it is accepted that changes in length are closely accompanied by changes in weight, as has been shown by R. E. Davis and G. E. Troxell[6], then the expectation would be to have the greater volume changes for the mixes with the highest water-cement ratio since they showed the highest absorptions. This in general was true.

TABLE 10—48 HOUR ABSORPTIONS OBTAINED AFTER SPECIMENS HAD BEEN SUBJECTED TO DESIGNATED NUMBER OF CYCLES

Absorptions Given in Pounds. Each Specimen Weighed About 23.5 Lb.

Cement	Mix	Vibrated Mixes						Mix	Hand-Rodded Mixes					
		Absorptions After Cycle No.							Absorptions After Cycle No.					
		8	10	20	40	60	90		8	10	20	40	60	90
1	A	0.16		0.13	0.10		0.12	E	0.19		0.11	0.09		0.10
1	B	0.22		0.15	0.12		0.12	F	0.20		0.16	0.16		0.12
1	C	0.25		0.21	0.19		0.15	G	0.28		0.23	0.25		0.14
1	D	0.28		0.22	0.20		0.16	H	0.35		0.29	0.33		0.19
1	J	0.42		0.38	0.38		0.35	K	0.42		0.40	0.40		0.37
2	A	0.18		0.11	0.08		0.11	E	0.16		0.08	0.05		0.10
2	B	0.15		0.08	0.05		0.09	F	0.23		0.12	0.08		0.08
2	C	0.20		0.14	0.12		0.12	G	0.24		0.18	0.15		0.09
2	D	0.31		0.21	0.16		0.14	H	0.30		0.22	0.23		0.13
2	J	0.35		0.21	0.22		0.20	K	0.39		0.38	0.36		0.30
3	A	0.18		0.11	0.11		0.09	E	0.18		0.09	0.07		0.08
3	B	0.23		0.15	0.16		0.11	F	0.19		0.14	0.12		0.14
3	C	0.27		0.18	0.22		0.13	G	0.21		0.16	0.17		0.18
3	D	0.32		0.23	0.32		0.21	H	0.22		0.18	0.20		0.23
3	J	0.37		0.30	0.33		0.34	K	0.30		0.23	0.30		0.30
4	A		0.13			0.10		E		0.15			0.15	
4	B		0.14			0.15		F		0.16			0.14	
4	C		0.16			0.17		G		0.17			0.15	
4	D		0.21			0.26		H		0.22			0.20	
4	J		0.22			0.28		K		0.22			0.25	
5	A		0.06					E		0.05				
5	B		0.07					F		0.07				
5	C		0.09					G		0.06				
5	D		0.10					H		0.09				
5	J		0.10					K		0.10				

RESULTS OF TESTS—SERIES C

General Considerations—The accumulated data on plastic flow have been obtained for mixes of about two to seven inch slump, placed by hand. Because of the increasing importance of the vibrator as a means of placing concrete and because of the lack of published data on the plastic flow of vibrated concrete, it was decided to investigate this variable.

Plastic flow as determined in this work is the axial deformation due to constant sustained load. It is the difference between the total deformation at any age, corrected for length changes due to shrinkage and variation in temperature and humidity which are not a part of the plastic flow deformation, and the instantaneous deformation which takes place immediately upon application of the load. The above method of calculation of plastic flow is fundamentally the same as that given by Raymond E. Davis and Harmer E. Davis.[7] The change in the modulus of elasticity of the concrete with age and the creep during loading, factors considered by W. H. Glanville[8], have been disregarded.

Relation of Modulus of Elasticity, Mix Proportions, and Method of Placement—The data presented in Table 11 give values of secant modulus of elasticity determined at three unit stresses expressed as fixed proportions of the ultimate unit compressive stress. The data show in all cases for a given concrete that the secant modulus of elasticity decreases as the unit stress at which the determination is made is increased.

For mixes made with the same water-cement ratio and the same cement the modulus of elasticity values are usually greater for the vibrated mixes. There is also a tendency for the modulus of elasticity to decrease as the water-cement ratio increases for both the hand-rodded and the vibrated mixes. The values for mix G1 do not agree with the general trend and appear to be higher than would be expected. The type of cement appears to have little effect, with only a slight superiority shown for the mixes made with cement 4. It should also be noted that for the leanest vibrated mix, J, the modulus of elasticity values have not suffered great numerical decreases from the values as given for the richest vibrated mix, A, indicating that the changes in modulus of elasticity with changes in mix proportions or water-cement ratio are not as great as strength or permeability changes.

Comparison of Modulus of Elasticity Values Obtained by Strain Gage and by Compressometer—The comparison in Table 11 has been tabulated for a stress 25 per cent of the unit compressive ultimate for each mix, and with one exception, G1, shows that the modulus of elasticity as determined with the strain gage was lower than that determined with the compressometer. The average difference between the strain gage and compressometer values was about 10 per cent.

The most probable reason for this difference is the greater short time creep or flow in the strain gage tests. Because of the time required to take strain gage readings it is estimated that the strain gage deter-

TABLE 11—VALUES OF MODULUS OF ELASTICITY AND POISSON'S RATIO

Mix	Mix Proportions, By Weight	Cement	Method of Placement	Water-Cement Ratio, Weight	Secant Modulus of Elasticity* at			Modulus of Elasticity‡ at 0.25 f'_c as determined by		Poisson's Ratio‡ at		
					0.22f'_c	0.25f'_c	0.33f'_c	Compresso-meter	Strain Gage	0.22f'_c	0.25f'_c	0.33f'_c
A	1:3:7	1	Vib.	0.491	4,110,000	4,050,000	3,920,000	4,340,000	3,760,000	0.163	0.163	0.164
		2			4,470,000	4,400,000	4,100,000	4,540,000	4,260,000	.....	.....	.....
		4			4,480,000	4,430,000	4,370,000	4,540,000	4,320,000	0.167	0.167	0.168
C	1:4½:9½	1	Vib.	0.660	4,020,000	3,850,000	3,630,000	4,340,000	3,360,000	0.161	0.161	0.159
		2			4,160,000	4,070,000	3,950,000	4,200,000	3,940,000	0.165	0.167	0.171
		4			4,330,000	4,280,000	4,170,000	4,560,000	4,000,000	0.156	0.158	0.159
J	1:6:12	1	Vib.	0.850	4,260,000	4,040,000	3,560,000	4,160,000	3,920,000	0.122	0.122	0.124
		2			4,220,000	4,000,000	3,680,000	4,420,000	3,590,000	0.127	0.120	0.112
		4			4,280,000	4,150,000	3,880,000	4,510,000	3,780,000	0.166	0.166	0.165
E	1:2:4½	1	Hand	0.491	3,880,000	3,810,000	3,660,000	3,910,000	3,710,000	0.174	0.175	0.175
		2			4,200,000	4,170,000	4,120,000	4,360,000	3,980,000	0.178	0.179	0.181
		4			3,860,000	3,850,000	3,760,000	4,030,000	3,660,000	0.161	0.162	0.162
G	1:3½:6	1	Hand	0.660	4,340,000	4,200,000	3,850,000	4,050,000	4,350,000	0.154	0.151	0.161
		2			3,700,000	3,650,000	3,560,000	3,870,000	3,430,000	0.159	0.160	0.164
		4			3,960,000	3,890,000	3,730,000	4,040,000	3,730,000	0.159	0.159	0.158

*Each is the average of four determinations, two by strain gage and two by compressometer.
‡Average for two specimens in each case.

mination required about twice the time necessary for the compressometer determination. That concrete will creep under load and that the actual values for a load duration of less than 1 minute are significantly large has been shown by W. H. Glanville[8], P. G. Jones, and F. E. Richart[9]. Consequently it is believed that because of the longer test period for the strain gage determinations the resultant creep produced larger strains than obtained by compressometer for any given load, resulting in a lower modulus of elasticity.

The presence of creep over a period of 1½ hours is definitely shown in this work by a comparison of the modulus of resistance after 1½ hours, determined by strain gage, Fig. 15, with the values of the modulus of elasticity as determined by strain gage at 25 per cent of the ultimate compressive strength, Table 11. The modulus of resistance is used here as by Raymond E. Davis and Harmer E. Davis[7], namely—ratio of unit sustained stress to total unit deformation, including instantaneous deformation and flow. It is noted that in all cases, except A4 and E4, the modulus of resistance after 1½ hours is less than the instantaneous modulus of elasticity, and this decrease is undoubtedly due to creep action. In the case of A4 the modulus of resistance given is for a stress 17.6 per cent of the compressive ultimate, and this value would have been decreased if the determination had been made at 25 per cent so that it is probable that this value is in line with the others.

Relation of Poisson's Ratio, Mix proportions, and Method of Placement—The data in Table 11 show that Poisson's ratio is practically constant over the range of stress from 22 to 33 per cent of the ultimate compressive strength. There is no definite trend shown, at least for the range of stress indicated. There is a tendency for Poisson's ratio to decrease as the water-cement ratio is increased, for both the vibrated and the hand-rodded mixes made with cements 1 and 2 but the trend is not definite for the mixes made with cement 4. The largest value of Poisson's Ratio, 0.181, was obtained with mix E2, while the lowest value, 0.112, with mix J2. There was no definite difference between the vibrated and hand-rodded values for mixes of the same water-cement ratio and made with the same cement. Poisson's Ratio, as the modulus of elasticity, did not change as much with changes in water-cement ratio or mix proportions as did the strength or permeability.

Relation of Plastic Flow, Water-Cement Ratio, and Method of Placement—The data of Table 12 give the loads on the specimens in terms of actual unit stress and in per cent of the compressive ultimate

TABLE 12—RELATION BETWEEN SPECIMEN LOAD, INSTANTANEOUS DEFORMATION PLUS 1½ HOUR PLASTIC DEFORMATION, AND PLASTIC DEFORMATION AFTER CERTAIN AGES*

Mix	Mix Proportions, by Weight	Water-Cement Ratio, by Weight	Specimens Loaded to		Instantaneous Deformation Plus 1½ Hr. Plastic Deformation, Per Cent	Plastic Deformation After: per cent			Shrinkage Deformation After: per cent	
			Per Cent Ult.	Unit Stress Lb. per Sq. In.		200 Days	1 Year	2 Years	28 Days	2 Years
A1	1:3:7	0.491	.250	1000	.0268	.0295	.0325	.0370	.0100	.0320
A2			.211	1220	.0293	.0325	.0355	.0407	.0150	.0350
A4			.176	1220	.0275	.0235	.0270	.0325	.0120	.0400
C1	1:4¼:9½	0.660	.250	570	.0193	.0200	.0220	.0255	.0120	.0325
C2			.250	1120	.0330	.0330	.0340	.0385	.0178	.0348
C4			.250	1210	.0320	.0310	.0385	.0455	.0080	.0370
J1	1:6:12	0.850	.250	380	.0132	.0280	.0300	.0335	.0068	.0288
J2			.250	660	.0185	.0260	.0280	.0305	.0112	.0275
J4			.250	640	.0200	.0435	.0535	.0610	.0018	.0352
E1	1:2:4½	0.491	.250	830	.0258	.0525	.0565	.0623	.0122	.0402
E2			.217	1220	.0320	.0505	.0540	.0600	.0180	.0452
E4			.250	1160	.0305	.0320	.0370	.0470	.0162	.0560
G1	1:3½:6	0.660	.250	450	.0115	.0260	.0275	.0335	.0118	.0405
G2			.250	940	.0300	.0665	.0720	.0770	.0212	.0478
G4			.250	1030	.0310	.0450	.0555	.0715	.0135	.0505

*Values given are the average for 2 specimens.

strength, the instantaneous deformation plus 1½ hours plastic flow, and also the plastic flow after certain ages. It should be noted that all specimens with the exception of A2, A4, and E2, were loaded to 25 per cent of their ultimate. In the case of the three exceptions the maximum spring load was applied which resulted in stresses of 21.1, 17.6, and 21.7 per cent of the ultimate respectively. Loading the specimens to 25 per cent of the ultimate strengths produced variable unit stresses, which varied from a maximum of 1220 p.s.i. to a minimum of 380 p.s.i. The vibrated specimens for a given water-cement ratio and a given cement were usually subjected to a slightly higher stress than the corresponding hand-rodded specimens. The data show that plastic flow after 200 days and the instantaneous deformation plus 1½ hour plastic flow for the vibrated mixes A and C were approximately the same, but that for the leanest vibrated mix, J, and for the two hand-rodded mixes E, G, the plastic flow after 200 days was the greatest.

The actual plastic flow curves for the specimens are shown in Fig. 14. These curves show very conclusively that in specimens loaded to a definite percentage of their ultimate strength and for a given water-cement ratio and a given cement the hand-rodded mixes had greater plastic flows in all cases. Because of the different actual unit stresses imposed on the loaded specimens it is difficult to make comparisons as to the effect of water-cement ratio or cement type on plastic flow.

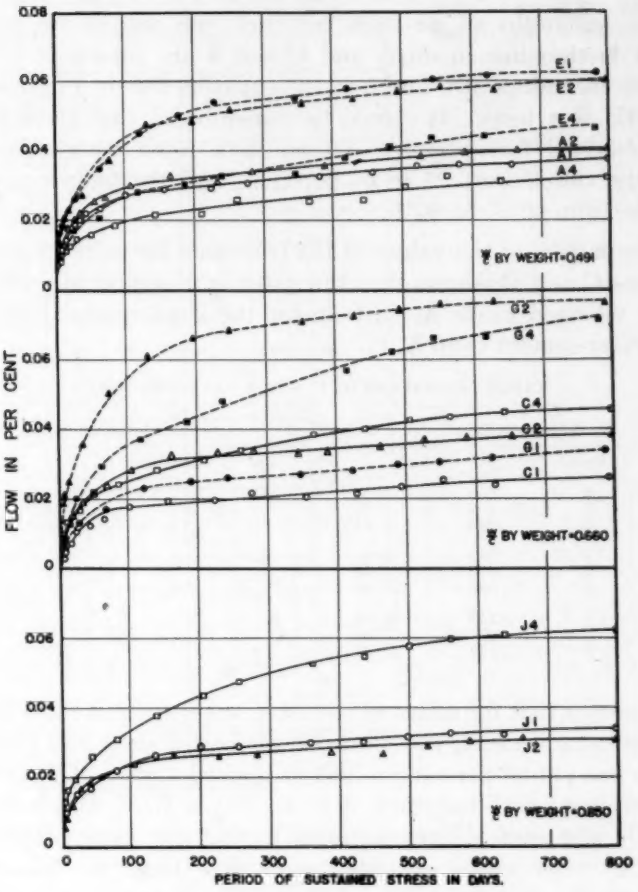


FIG. 14—PLASTIC FLOW CURVES

Mixes made with cement 4 show the greatest tendency for plastic flow to increase after two years.

The plastic flow curves plotted on log-log paper were straight lines for approximately the first two hundred days of sustained loading for the specimens made with cements 1 and 2, and for the entire period of observation, two years, for the specimens made with cement 4. The equation of each was determined and found to be of the form

$$Y = Cx^{1/a} \text{ or } Y = C\sqrt[a]{x}$$

This equation is the same one used by J. R. Shank[10] in his analysis of existing plastic flow data. In this equation Y is the plastic

flow in millionths of an inch per inch per one pound per square inch, x is the time in days, and C and a are constants. Table 13 presents the values of C and a in the equation for the mixes subjected to plastic flow tests. It should be remembered that these constants were obtained for specimens stored in a room where the relative humidity varied from 25 to 60 per cent, and the temperature variation was from 60°F. to 80°F.

A comparison of the values of the constants for mixes A and E, also for mixes C and G, shows that the value of Y will in all cases be less for the vibrated mixes A, C, than for the hand-rodded mixes of the same water-cement ratio E, G.

TABLE 13—VALUES OF C AND a FOR GIVEN MIXES

Vibrated Mixes				Hand-Rodded Mixes			
Mix	Cement	C	a	Mix	Cement	C	a
A	1	.056	3.00	E	1	.105	3.00
	2	.074	4.00		2	.073	3.00
	4	.049	3.65		4	.053	3.25
C	1	.060	3.00	G	1	.100	3.00
	2	.050	3.00		2	.120	3.00
	4	.057	3.55		4	.058	2.70
J	1	.170	3.65				
	2	.098	3.65				
	4	.120	3.20				

This means that for mixes of the same water-cement ratio and made with the same cement, the vibrated mixes will show less plastic flow for each one pound per square inch of applied stress. This conclusion is in agreement with the work of R. E. Davis, H. E. Davis and E. H. Brown[11] who state "For concretes having the same water-cement ratio, the richer mixes exhibited more flow than the leaner mixes. This is what might be expected if the major portion of the flow is considered to be due to movement in the hardened paste." The hand-rodded mixes had more paste than did the vibrated mixes of the same water-cement ratio and this may be considered the reason for the different plastic flows obtained. The above conclusion points to the fact that vibrated concrete will not allow the stress redistribution due to plastic flow that hand-rodded concrete will, and consequently the vibrated concrete may show considerably more cracking under similar external load conditions.

Subsequent to the discussion of R. E. Davis regarding the greater possibility of shrinkage cracks in the hand-rodded concrete because of higher cement contents the shrinkage data in Table 12 have been added. The two-year shrinkage data show that for a given water-cement ratio and a given cement the hand-rodded concrete did shrink

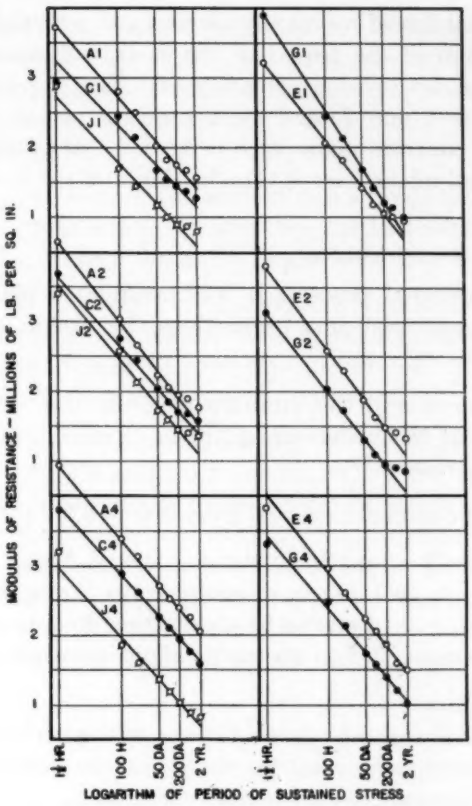


FIG. 15—RELATION OF MODULUS OF RESISTANCE TO LOGARITHM OF PERIOD OF SUSTAINED STRESS

considerably more than the vibrated concrete, thus confirming his remarks.

Relation of Modulus of Resistance to Period of Sustained Stress—Fig. 15 shows the relation of modulus of resistance to period of sustained stress. It shows that the modulus of resistance in all instances decreases with the period of sustained stress, and that the decrease is greatest during the early part of the period. After two years of sustained stress the largest value of the modulus of resistance was 2,030,000 p.s.i. for mix A4, while the lowest was 790,000 p.s.i. for mix J4. The data show that the modulus of resistance after two years sustained loading varied between one-half to one-fifth of the initial secant modulus of elasticity. This change in modulus of resistance is important in reinforced concrete columns, beams and arches, where the change in the modulus of resistance will result in a redistribution of the dead load stresses, giving higher steel stresses and lower concrete stresses than those based on initial secant modulus of elasticity.

Fig. 15 shows that the modulus of resistance decreases as a straight line function of the logarithm of the period of the sustained stress. This relationship holds true up to approximately 200 days for the concrete made with cements 1 and 2, and for the entire period of observation, two years, for concrete made with cement 4. It is also interesting to note that for all mixes, except G1, the slopes of the lines are all nearly the same.

CONCLUSIONS

1. With the aid of the internal vibrator it was possible to place satisfactorily no-slump concrete with mix proportions, by weight, of 1:6:12.

2. For both the hand-placed and the vibrated concrete the compressive strength decreased as the water-cement ratio increased, and as the cement-voids ratio decreased.

3. The 28-day compressive strengths varied from 990 to 6300 p.s.i.

4. At 28 days the high early strength portland cements, 2 and 5, and the high silica cement, 4, had the best compressive strengths. The normal portland cement, 1, had strengths about three-fourths as great, while the low heat cement, 3, has strengths about one-half as great.

5. For both the hand-placed and the vibrated concrete the permeability increased as the water-cement ratio increased, as the cement content decreased, and as the cement-voids ratio decreased.

6. For a given cement and water-cement ratio the vibrated concrete was from 125 to 1065 p.s.i. stronger than the hand-placed concrete, and in general was also less permeable.

7. The type of cement used had a large effect on permeability, with the greatest differences evident in the high water-cement ratio mixes. Concrete made with the high silica cement and high early strength portland cement, 5, was relatively impermeable, while that made with the normal portland cement and the low heat cement, especially the leaner mixes, was very permeable.

8. The vibrated mixes generally had more air voids than did the hand-placed concrete for a given water-cement ratio. The air voids by absolute volume for the various vibrated compression mixes varied from .017 for the mix with the lowest water-cement ratio to .041 for the mix with the highest water-cement ratio, while for the hand-rodded mixes the variation was from .011 to .039.

9. The specific weight of the vibrated and hand-rodded specimens decreased about 4 lb. per cu. ft. as the water-cement ratio was varied from 0.491 to 0.850, and for a given water-cement ratio the vibrated cylinders weighed about 4 lb. per cu. ft. more than the hand-rodded cylinders.

10. For a given cement and a given cement voids ratio the vibrated concrete was approximately 200 to 1600 p.s.i. stronger than the hand placed concrete, and was also generally less permeable.

11. Properly cured concrete with a 28-day compressive strength of 3000 p.s.i. or better was only slightly permeable, regardless of method of placement or type of cement used. For lower strengths the type of cement used was very important.

12. The cement content for the vibrated mixes varied from about 2.2 to 3.9 sacks per cu. yd., while the variation for the hand-placed mixes was from 2.7 to 5.5 sacks per cu. yd. Comparisons for given cement types and contents showed that the vibrated cylinders were approximately 600 to 3000 p.s.i. stronger. Similar comparisons showed that the vibrated cylinders had inflows of water approximately 100 to 170,000 millionths of a gal. per sq. ft. per hr. lower than those for the hand-rodded cylinders.

13. Concrete made with the high early strength portland cements had larger 28-day air shrinkages than did those made with the other types of cements. After two years the concrete made with the high silica cement also showed large shrinkages.

14. Repeated heating and water soaking of the concrete resulted in increasing expansions. The expansions of the concrete cylinders made with cements 1, 2, and 3 increased, in general, as the water-cement ratios increased, but there was no definite difference in the values for the vibrated as against the hand-placed concrete.

15. Mixes made with the high early strength portland cement 2, and with the high silica cement showed the lowest expansions, while those made with the normal portland cement showed the highest.

16. Mixes made with the two high early strength portland cements, and with the high silica cement showed relatively low absorptions, while the mixes made with the normal portland cement showed high absorptions.

17. For a given concrete the secant modulus of elasticity decreased as the unit stress at which it was determined was increased; but Poisson's ratio was practically independent of the stress at which it

was determined, for stresses between 22 to 33 per cent of the compressive ultimate.

18. There was a tendency for the secant modulus of elasticity and Poisson's ratio to decrease as the water-cement ratio increased, and for a given water-cement ratio vibration had no material effect on Poisson's ratio but increased slightly the modulus of elasticity.

19. Neither the modulus of elasticity nor Poisson's ratio appeared to vary with the type of cement used.

20. Values of Poisson's ratio and modulus of elasticity did not vary as much with changes in the water-cement ratio as did the compressive strengths and permeabilities.

21. The secant modulus of elasticity values as determined by compressometer averaged about 10 per cent higher than the values determined by strain gage. This difference is believed due to greater flow of the specimens in the strain gage tests because of a necessarily longer test period.

22. Regardless of method of placement, type of cement, mix proportions, or water-cement ratio, the concrete subjected to sustained compressive stress deformed plastically over the period of observation, about two years. The plastic flow values after two years of sustained loading varied from .0255 to .0770 per cent.

23. The plastic deformations under constant load are large at first and decrease as the period of loading is increased indicating that a no-flow condition probably will result after a comparatively long period of loading. Approximately 50 per cent of the plastic flow obtained after two years of sustained loading took place within the first two months.

24. All of the plastic flow-time curves for the first 200 days had equations of the form $Y = C^{a}\sqrt{x}$. The constant C varied between .050 to .170 and the constant a varied between 2.70 to 4.00.

25. For specimens loaded to 25 per cent of their ultimate compressive strengths and for a given water-cement ratio, the hand-rodded mixes showed greater plastic flows than did the vibrated mixes. After two years of sustained loading, the hand-rodded-concrete specimens had plastic flows from 1.3 to 2.0 times as great as the vibrated specimens. The hand-placed concrete, however, had two year shrinkage values from 1.25 to 1.4 times as great as those for the vibrated concrete.

26. The modulus of resistance, at least up to a period of sustained stress of 200 days, decreased as the logarithm of the period of sustained

stress increased. The modulus of resistance after two years of sustained stress varied between 790,000 and 2,030,000 p.s.i.

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Discussion, to close in the September Supplement, should reach A. C. I. Secretary by July 20.

Vol. 36 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE
(Supplement)

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *SEPTEMBER, 1940*

Discussion of a paper by George W. Washa:

Comparison of the Physical and Mechanical Properties of Hand Rodded and Vibrated Concrete Made With Different Cements*

BY C. H. SCHOLER, F. E. RICHART, RAYMOND E. DAVIS, M. O. WITHEY, T. C. POWERS, WILLIAM R. LORMAN AND AUTHOR'S CLOSURE

CONVENTION DISCUSSION

C. H. Scholer†—It is not clear to me how silica cement or cement with a silica addition ground in, is made.

Mr. Washa—The high silica cement was made by grinding together 70 per cent portland cement clinker and 30 per cent high silica compound.

F. E. Richart‡—It has been stated that plastic flow with the vibrated specimens is much less than with the rodded. I think this is of interest, because the same thing has been observed and recorded in England—the fact that vibrated concrete shows less flow than other concrete may be a disadvantage in some cases, and I think it was so mentioned by Dr. Glanville, who found that because of the inability to flow, vibrated concrete in some cases in slabs was subjected to severe cracking, where the concrete which was not vibrated did not crack so badly. This property which we call plastic flow is in many cases a beneficial property and the fact that vibrated concrete may have less of it than other concrete, may in some cases be a disadvantage.

*Raymond E. Davis***—Following Professor Richart's remarks it should be pointed out that the vibrated concrete with its lower cement content would shrink less due to drying than the concrete which had

*JOURNAL, Amer. Concrete Inst., June 1940; *Proceedings* Vol. 36, p. 617.
†Binghampton, N. Y.
‡University of Illinois.
**Berkeley, Cal.

been hand-rodded. One effect at least partially offsets the other, since we recognize that most of our cracks are due primarily to shrinkage. I thought that should be brought out here. For the leaner mixes of the vibrated concrete we would naturally expect the flow to be less, but we would also expect the shrinkage to be less.

Question—I should like to ask just how the plastic flow tests were made?

Mr. Washa—These plastic flow tests were made by making up six specimens for each variable; two of them were broken to determine the 28-day compressive strength; two were placed in the plastic flow assemblies and subjected to a sustained stress of one-quarter of the ultimate compressive stress; the other two, previously pre-stressed, were taken with the two specimens in the assemblies and all were placed in one room. When the measurements were made the specimens in the assemblies and the control specimens were measured so that we could arrive at the deformation due simply to plastic flow and divorce from it changes due to temperature, humidity and shrinkage.

Mr. Washa—(in response to a question): The modulus of resistance is the ratio of the sustained unit stress to the total unit deformation, that is, the instantaneous deformation which is arrived at when the specimen is loaded, plus the plastic flow which is obtained after any certain period. As I see it, it simply indicates a change in the elastic properties of the material. That is, in our calculations we assume that we have a definite value of modulus of elasticity and that is obtained by dividing the working stress by the strain that accompanies a short time loading. If this specimen has been under load for any long period, this ratio, this value of E is no longer correct for dead load stress calculation.

*T. C. Powers**—It seems to me that the term modulus of resistance carries with it the implication that the elastic properties of the concrete had changed as indicated by that ratio. I believe that what Mr. Washa has just said seems to bear out that interpretation. However if we were to determine the modulus of elasticity at any stage of this loading, it would be found to be higher than the modulus before loading; in other words, the response of that concrete at any given time to a change in stress would be indicated by a modulus of elasticity comparable to the original, plus whatever age effect there might be. The so-called sustained modulus that we get is not to be construed as an elastic modulus.

Mr. Washa—That is a fact.

*Portland Cement Assn., Chicago.

*M. O. Withey**—I should like to ask Mr. Powers how he would calculate the stress after a year on a specimen showing a total unit change of two ten-thousandths of an inch?

Mr. Powers—Perhaps such a factor (sustained modulus) is practical to use; the issue that I am raising concerns the erroneous idea that the elastic properties of the concrete have changed as indicated, for that idea can give rise to erroneous thinking and conclusions. The practical matter of computing the stress in a beam from a measured deformation seems to me to be something else.

BY WILLIAM R. LORMAN†

It is unfortunate that some of the series C test specimens were prestressed to one-third of their ultimate strength and then later employed as control specimens in the creep tests. Control specimens, to be considered as truly such, should not be subjected to any loads whatsoever. The writer therefore believes that the creep data obtained are probably subject to some misgiving.

Although all creep specimens during storage were subjected to the same temperature range (60°F. to 80°F.) and to the same variations in relative humidity (25 to 60 per cent) a preferable procedure would have been to use standard storage conditions such as constant temperature of 70°F. and 100 per cent relative humidity. Humidity and temperature during the loaded period are important physical factors affecting the values of the creep coefficient m which has been defined[4]* as the ultimate creep strain per unit of stress. Consideration of the gel theory indicates that humidity during the loaded period affects creep only inasmuch as it influences expulsion of moisture from the concrete. For example, at 70 per cent relative humidity the ultimate creep is generally twice that at 100 per cent relative humidity.

Regarding the data of Table 12 and Fig. 14 the following comments are submitted. From test results published by Glanville[3] and by Ross[6] it appears that Hooke's law applies to creep strain accurately for stresses ranging from 0 to 650 p.s.i. and approximately for stresses ranging from 650 to 1500 p.s.i. Specimens fabricated from mixes A1, A4, E1, and E4 were subjected to stresses considerably above 650 p.s.i. Thus the values of m and n listed below are more or less approximate.

Using the hyperbolic creep-time relation expounded by the writer[4], the following tabulation is presented:

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†Assistant Engineer, Bureau of Reclamation, Denver.
[4]See list of references.

Washa's mix number	A 1	C 1	J 1
W/C ratio by weight	0.491	0.660	0.850
Creep coefficient m	0.42	0.59	1.02
Creep-time constant n	100	200	100

This listing of creep data illustrates the variation of creep with water-cement ratio when the slump is constant. It is obvious that had more water-cement ratio values been employed in the experiments sufficient data would have been made available enabling the establishment of an empirical relation between water-cement ratio and creep coefficient m. The writer believes this relation to be of the form

$$_m = K\,(W/C)^2$$

where m = creep coefficient in inches x 10^{-6} per inch per 1 p.s.i.
W/C = water-cement ratio by weight.
K = a constant.

It is also believed that had the tests involved a dozen or more various water-cement ratio values, creep could be accurately represented as a function of the mix proportions using the contour method described by the writer[4]. Such comparison would probably demonstrate that water-cement ratio and not cement content is the determining factor regarding magnitude of creep.

In the following tabulation are shown values of creep constants for hand-rodded and for vibrated concretes both of identical water-cement ratio.

	(Mix E) Hand-rodded		(Mix A) Vibrated	
Type cement	m	n	m	n
Normal portland	0.86	100	0.42	100
Portland-puzzolan	0.48	160	0.32	160

These values serve to demonstrate that vibration compacting decreases creep but does not affect the time, in days, at which one-half the ultimate creep is attained. The comparison also serves to indicate the effect of type of cement upon the creep constants. However, regardless of type of compaction, the relative values of m (on the basis of type of cement) apparently are not in agreement with facts observed in the field. Field experience has repeatedly demonstrated that mass concrete made with low-heat or portland-puzzolan cement (typical special cements) resists cracking to a better degree than concrete made with normal portland cement. Test results described by Savage[5] indicate that the superior resistance to cracking, exhibited by special cement concretes, is primarily due to greater ability to creep. Although the creep coefficient values tabulated above were obtained with concretes not mass-cured, certainly the effect of type of curing could not

possibly account for the entire reversal of creep properties indicated. It is believed that a portland-puzzolan cement concrete should result in more creep than a normal portland-cement concrete, regardless of type of compaction and type of curing. Since the data do not appear to substantiate this thought, prestressing of control specimens is a factor which conceivably would serve as an explanation of the disagreement with well established fact.

Compaction should be regarded in terms of frequency and acceleration. In this connection reference should be made to the work of the British Building Research Board.[1,2] When sufficient data have been acquired, varying throughout the maximum permissible range of acceleration and frequency, then and then only can an accurate idea be obtained regarding the effect of vibration upon the creep constants of a concrete.

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AUTHOR'S CLOSURE

In my answer to the question raised by Mr. Powers, it is unfortunate that the term "elastic properties" was used. It would have been better to have said "stress-strain properties".

Referring to Mr. Lorman's subsequent written discussion, the primary purpose of the plastic flow tests was to compare the effects of the two types of placement, consequently the only essential storage condition was that both types be subjected to the same conditions during test. The storage condition chosen was one that simulated conditions which concrete placed in interior building construction would undergo, thus including the variable effects of changes in temperature and humidity.

In the discussion on the effect of type of cement on plastic flow properties the low heat and high silica cements have been considered together. Since there was a very large difference in composition and

properties between the low heat and the high silica portland cements used in these tests it would also be reasonable to expect a considerable difference in behavior of the two types in plastic flow tests. It is unfortunate that time and facilities did not allow plastic flow tests of the low heat cement to be made.

In comparing the plastic flow data obtained for the various cements with published data it is believed that the following items should be considered:

(1) The specimens were stored during test under variable temperature and humidity conditions, as contrasted to a generally fixed condition.

(2) Any comparison of values should be based on similar storage conditions prior to test.

(3) It does not seem possible that prestressing of the control specimens would cause relatively greater changes in the plastic flow of concrete made with one type of cement over that made with another type. The author agrees however that the use of unstressed control specimens would have obviated the question raised by Mr. Lorman.

(4) In Mr. Lorman's discussion of the plastic flow of concrete made with high silica and low heat cements some consideration should be given to the effect of cement fineness. The specific surface of the high silica cement used was 2075 sq. cm. per gram which is considerably higher than that found in coarsely ground low heat cements. Plastic flow of concrete made with this type of cement decreases rapidly as the specific surface is increased.

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The Application of Some of the Newer Concepts to the Design of Concrete Mixes*

By W. M. Dunagan†
MEMBER AMERICAN CONCRETE ISTITUTE

SYNOPSIS

Present day concrete proportioning practice can be benefited by (1) an "up to date," simple and reasonably accurate method for approximating mixes before work starts so that required materials may be estimated and so that a guide is furnished for the first "trial mix," and (2) a computation method to facilitate mix adjustment in the interest of workability, during the progress of the work. Good specifications are based upon such a procedure. Two factors have hindered the full provision of these two details: First, no system has provided a direct approximation of the water necessary for workability; second, the rules governing the balance between fine and coarse aggregate have not been clear; these have been assumed as discoverable only by trial. This author has developed by test and condensed into diagrams for this paper a means for approximating the first item (the water requirements of a mix) and has used a formula developed by C. A. G. Weymouth[1] to provide a clue for a discovery of the second (the fine-coarse balance). He uses these data in conjunction with procedures taken from other authors to develop a complete system for designing concrete mixes.

The author states his system for designing mixes in an eight point outline citing precedent for the use of the main principles. In the body of the paper he develops the use of individual items and submits data to show the accuracy which may be expected. The paper concludes with an appendix in which typical problems are solved.

INTRODUCTION

The very much alive problem twins—grading and proportioning—will submit themselves to satisfactory solution only when they are solved by methods so basic in concept that the significance of every

*Presented 36th Annual Convention American Concrete Institute, Chicago, Feb. 27-29, 1940.
†Assoc. Prof., T. & A. M. Dept., Iowa State College, Ames, Iowa. Test data taken as part of Project 230, Eng. Exp. Sta.
(1)See list of references.

step in the solution is clear to the user and leads him directly to practical results. Concrete proportioning knowledge is rich in details which contribute to an understanding of these problems but they have not been made to fit into a common, understandable scheme. This paper is an effort to reconcile some of the newer details with some of the older concepts to evolve a unified approach, and to secure by tests certain information needed to make the resulting procedure practical. The result is a rather general but basic solution for the old problem "proportioning concrete mixes"; it advances the solution of these problems by demonstrating how certain of the older ideas and more recently announced theories do fit into a common picture.

The specific developments here brought together are*:

(1) Abram's practical use[2] of the principle that the quality of concrete may be governed by control of the ratio of water to cement.

(2) McMillan's demonstration[3] that when the ingredients of a concrete mixture are discussed in their absolute volumes† (percentages per unit volume of concrete) complexities are eliminated and thinking is systematized, and his insistence upon the importance of the "paste of water and cement" concept.

(3) Slater's and Lyse's rule[4] that "for a given aggregate the water required for a given consistency per unit volume of concrete is constant for all cement contents."

(4) Talbot's and Richart's use of the principle[5] that for a given aggregate the absolute volume of coarse aggregate in a mix for a given consistency may properly be held constant for all cement contents.

(5) C. A. G. Weymouth's suggestion[1] to space aggregate in concrete according to a "particle interference" criterion, with his formula for evaluating the spacing in terms of the factors which govern it.‡

OUTLINE OF THE AUTHOR'S DESIGN AND ADJUSTMENT PROCEDURE

This paper is intended to show how the five foregoing principles are assembled to form a unified scheme to design or to adjust concrete mixes. The scheme contemplates the design of mixes as consisting essentially of eight steps:

(1) The quality of concrete is considered to be established by the choice of the water cement ratio. Fundamentally compressive strength

*These developments are here used so broadly that it should not be necessary for the reader to go to the references in order to make the application of them intended by this paper. Their former use is cited to substantiate the present application.

†The use of absolute volumes in concrete problems did not originate with McMillan; he did, however, use them to develop a useful scheme for showing the components of mixtures. Figure 1b follows his scheme and the author's entire procedure is based upon the treatment of the ingredients in such terms.

‡Weymouth's basic concept is that "the aggregate of a given size forms a planar grid through which all materials of a smaller size must pass if concrete is to be workable." Strict agreement with his concept is not necessary to the use of his formula as is done in this paper.

is related to the quality of the paste of water and cement; this may be expressed in any terms; its expression as a ratio of cement to water by absolute volumes not only simplifies computations but gives the clearest basic picture. Convenient graphic presentation of this relationship may show the ratios of water to cement in such terms as gallons per bag or by weights interpolated in their appropriate positions. (See Fig. 1 and Fig. 2.)

(2) Slump in inches may be stated to limit the wetness of concrete but is not considered to be other than a rather crude measure of workability. A choice of workability is understood to imply placeability. (In this paper placeability is measured by remolding effort.)

(3) The amount of water, w, per unit volume of concrete that will be required for the chosen workability must be estimated. (See Fig. 10.)

(4) The cement content, c, per unit volume of concrete is computed as being a product: $c = (c/w)\ (w)$.

(5) The maximum amount of coarse aggregate per unit volume of concrete, b, that may be used commensurate with the desired workability is estimated in terms of the maximum size and type of the aggregates. This value may be found from tests and computations as explained in Table 1, or it may be approximated from Fig. 9.

(6) The amount of fine aggregate per unit volume of concrete, a, is found by difference; assuming that the air voids will be negligible, a, the fine aggregate, will equal $1 - (w + c + b)$.

(7) W, c, b and a are converted to weights of materials per unit volume of concrete by the method of Table 8; they may be converted to other terms as required by a particular situation.

(8) The first use of such an estimated mix is considered to be a trial mix. As a result of this trial the estimated values for w, the water, and for b the coarse aggregate may be modified. Holding constant the w and b values shown satisfactory in the trial the entire mix may then be adjusted so as to contain cement and water in the ratio which will produce the required quality, the sand, a, again being found by difference.

DEFINITION AND DISCUSSION OF TERMS USED

A cross section of a piece of concrete may be generalized as is done in Fig. 1a. McMillan[3] reduced this complex picture to one such as that shown in Fig. 1b. The author's system is based entirely upon ingredients expressed as percentages per unit volume of concrete with the parts named according to a terminology adopted from Talbot and Richart[5]. The terms and formulas that will be found are:

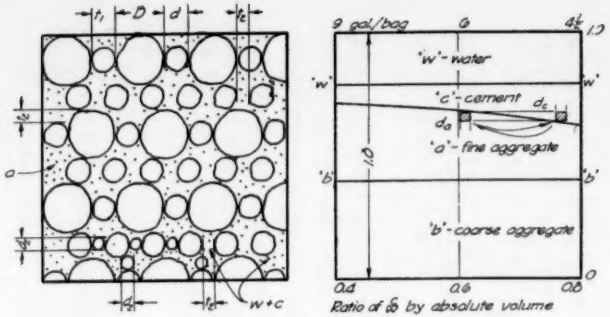


FIG. 1(A)—CONCRETE AS A MIXTURE OF INGREDIENTS

(a) The aggregate suspended in a paste of water and cement (w/c).
(b) Aggregate considered as being divided into size groups. The distance between the groups computed by Weymouth as being

$$"t" = [(do/da)^{1/3} - 1]D$$

(c) This paper points out how the Weymouth formula may be used to compute "b" the optimum coarse aggregate to use and how it may be used to set standards for grading.

FIG. 1(B)—INGREDIENTS SHOWN IN PARTS PER UNIT VOLUME OF CONCRETE (MCMILLAN). IN WHICH

$$b + a + c + w = 1.00$$

This paper points out the practical usefulness of combining the two assumptions:

(a) That the line w-w is horizontal.
(b) That the line b-b is horizontal.

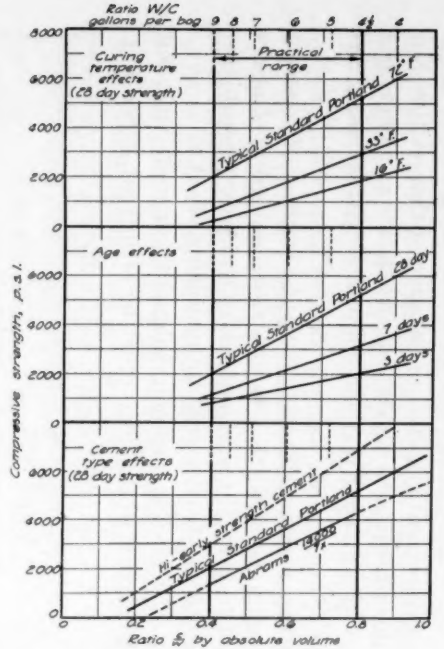


FIG. 2—RELATION BETWEEN THE RATIO $\frac{\text{CEMENT}}{\text{WATER}}, \frac{\text{C}}{\text{W}}$ BY ABSOLUTE VOLUMES AND THE COMPRESSIVE STRENGTH

Three other factors which influence the strength, cement type, age and curing temperatures are approximated on the same chart so that corrections may be made for them in a single operation. (This method for showing these relationships makes the computations of this author's methods more easily accomplished).

w = water content as its ratio to a unit volume of the concrete.
c = cement content as its ratio to a unit volume of the concrete.
a = fine aggregate content as its ratio to a unit volume of the concrete.
b = coarse aggregate content as its ratio to a unit volume of the concrete.

Voids due to the presence of air are assumed to be negligible. That this is a safe assumption is constantly borne out in practice by the correspondence between estimated and actual yield when the assumption is made that:

$$w + c + a + b = 1.00 \quad \ldots\ldots (1)$$

or that the sum of the absolute volumes of the components will equal the volume of the mixed concrete.

Inge Lyse[4] called attention to the usefulness of the fact that the line w-w in Fig. 1b may be assumed as horizontal. Data were secured in the researches for this paper to substantiate the relative accuracy of this assumption and to make certain distinctions necessary for its greatest usefulness.

w = a constant for constant workability, or that the line w-w in Fig. 1b is horizontal ..... (2)

Talbot and Richart[5] implied by making it an important part of their design procedure that b may be held constant for a given aggregate regardless of cement content. For use with their method b was established by determining bo, the density of the aggregate in question and using b as some fraction of bo. The choice of the ratio b/bo has been largely a matter of experience in the production of workable concrete with given aggregates.

b = a constant for constant workability, or that the line b-b in Fig. 1b is horizontal ..... (3)

(NOTE: Study of Fig. 1b will indicate that when assumptions (2) and (3) are made simultaneously the Slater-Lyse rule means that cement and fine aggregate may be interchanged in equal amounts by absolute volume without change in workability.)

Many writers have insisted that good grading is established when the size relationships conform to some definite system. Most of these systems have been based upon a grading curve of the type:

$$p = (d/D)^n \quad \ldots\ldots (4)$$

Where p = the amount of material in the system finer than size d.
d = the size of the particular group in question.
D = the largest particles used in the system.
n = an exponent governing the distribution of the sizes.

To date this formula has not been rationalized, exponents n have been arbitrarily assigned. This author's adaption of a formula by C. A. G. Weymouth is used to show how n may be determined for concrete gradings by test.

Weymouth[1] implied that for maximum workability with greatest economy the most desirable relationship between particle sizes (in Fig. 1a) should be one in which the spacing between particles is such that each has just sufficient "room" to move into the space between the particles of the next larger size. Assuming that the aggregates are divided into size groups by sieve standards so that each size group is one half the size of the next larger, then in Fig. 1a:

The diameter of particle d should equal distance $t1$ which should equal $0.5D$, and particle $d2$ should equal distance $t2$ which should equal $0.5d$, with this relationship extending throughout the entire mixture.

The maintenance of such a relationship becomes an ideal for "particle arrangement." To put this criterion into effect Weymouth showed the factors which govern it in a formula:

$$t = [(do/da)^{1/3} - 1]D \quad \ldots\ldots (5)$$

where t = the average distance between particles of an average diameter D.

do = the density of the size group. (The solids present in a unit volume alone, secured by a unit weight and specific gravity test.)

da = the ratio of the absolute volume of a size group to the space available to that size in the concrete.

THE RELATION OF WEYMOUTH'S FORMULA TO THE WORK OF THIS PAPER

This author made a study of the work of C. A. G. Weymouth to discover just what phenomenon in concrete might be explained in terms of "particle interference," (assuming that interference occurs at any time that t is less than $0.5D$). After a number of laboratory tests he was forced to the conclusion that particle interference as computed by the formula for t might be present in concrete without harmful effects. It computes as being present in concrete mixes of good design. When coarse aggregate is graded according to the straight line grading scheme of Fuller and Thompson[6] "interference" will occur in the largest sizes. Gap gradings are characterized by it. The harm is done when interference exists over a considerable portion of the grading curve. As will be shown in later figures recommended good practice such as that of H. N. Walsh[7], in which the coarse aggregate grading was derived from a straight line relationship, produces a spac-

ing for the largest size such that t for the next smaller is only $0.34D$; however, the next two smaller sizes are in such quantities that the defect is removed.

The fact that Mr. Weymouth's own methods for using this formula do not lend themselves to explanation of many strength and workability variations should not cause his formula to go unused or weaken his reasoning in regard to particle spacing in its stronger aspects. This author hopes to demonstrate a computation method for applying it in two such aspects, (1) to show that it may be used to compute b the limiting amount of coarse aggregate of a given type that may be used, which when used with the Talbot constant b assumption will automatically establish desirable fine-coarse aggregate balances and (2) that the Weymouth equation may be used as a powerful tool to establish criteria for good coarse aggregate grading.

USE OF THE WEYMOUTH EQUATION TO ESTABLISH LIMITING VALUES FOR b AND TO COMPUTE "IDEAL" GRADINGS

(NOTE: The computation method about to be shown will be carried through for one typical aggregate only. Results for four aggregates of widely varying types are developed in Table 6.)

When the aggregates are considered in size groups (shown as *a-b*, *b-c*, etc., in Table 1 and Fig. 5) divided by sieves such that each opening is one half that of the next larger and the spacing criterion is such that t equals $0.5D$, then in formula 5 the quantities in brackets must equal 0.50. For this to be true $(do/da)^{1/3}$ must equal 1.50, and for computation purposes the useful relationship is found that:

$$da = do/3.375$$

Now by definition da is the ratio of the absolute volume of material of a given size to the space available to that size, or in Table 1 it is Column (4) divided by Column (3). The amount of the largest size (*a-b*) must equal da, and if the density do of all size groups is the same the determination of the amounts of all sizes consists of computing those amounts that will keep da constant. With each da value a different amount of material will be found necessary to maintain a desired degree of particle freedom; and since da is derived from the value do which can be found experimentally, a given aggregate may be studied in terms of this factor.

To make practical use of these facts procede as follows: From a sample of the aggregate separate out a single size group and secure the value do by making a unit weight test and a specific gravity test of this size group. Having this value make calculations such as those of Table 1.

TABLE 1—THE USE OF THE WEYMOUTH FORMULA
$t = [(do/da)^{1/5} - 1]D$ to Determine b Values for a typical Well Rounded Pebble Aggregate

(a) From a unit weight test and sp. gr. of a size group..... $do = 0.63$
(b) For t to equal 0.5D da must be held constant for the value $do/3.375$..... $da = 0.186$
(c) Compute the absolute volume of each size per unit volume of concrete.

(1) Size group*	(2) "da"	(3) Space available to that group in the concrete	(4) Absolute volume of each size group per unit volume of concrete	(5) Sum of absolute volumes, S	(6) Maximum size at which $S = b$
a-b	0.186	1.000	0.186**	0.186	3/8"
b-c	0.186	0.814	0.151***	0.337	3/4"
c-d	0.186	0.665	0.123****	0.460	1 1/2"
d-e	0.186	0.540	0.100	0.560	3"
e-f	0.186	0.440	0.083	0.643	6"
f-g	0.186	0.337	0.066		

*Given letters because size groups descend by halves from the maximum which may be of any size.
**Must equal da since the space available to the largest size is 1.000.
***Computed by dividing 0.186 by (1.000—0.186).
****Computed by dividing 0.186 by (0.814—0.151).

The b values are found in Column (5). If the largest size, a-b, should be 3-1½ in., then there are four sizes larger than the No. 4 sieve and the sum of the first four computations in Column (4) is 0.56 which is b, the amount of this material that may be used for that maximum size; if a-b is 1½-¾ in. b is 0.46, etc. This is the method used to compute b introduced by this paper. Discovered by this method b will vary with do (which varies with the type of aggregate) and will vary with the maximum size of the aggregate as shown in Columns (5) and (6).

The data developed in Table 1 may then be used to produce "ideal" gradings for coarse aggregate in the following manner:

TABLE 2—IDEAL COARSE AGGREGATE GRADING PERCENTAGES IF THE BEST SPACING IS SUCH THAT $t = 0.5D$

Size group	For maximum size 3"			For maximum size 1½"			For maximum size ¾"		
	Ab. vol. for each group	Sum of ab. vol.	% each size	Ab. vol. for each group	Sum of ab. vol.	% each size	Ab. vol. for each group	Sum of ab. vol.	% each size
(1)	(2)*	(3)	(4)	(5)*	(6)	(7)	(8)*	(9)	(10)
3"—1½"	0.186	0.186	33.2						
1½"—¾"	0.151	0.337	26.9	0.186	0.186	40.4			
¾"—⅜"	0.123	0.460	22.0	0.151	0.337	32.8	0.196	0.186	55.1
⅜"—No. 4	0.100	0.560	17.9	0.123	0.460	26.8	0.151	0.337	44.9

*Taken from Column (4) Table 1.

Data developed as done in Column (4), Table 1, may be used to compute relative percentages in the manner of Table 2 for any number of aggregate sizes and used to form grading curves over which the spacing will be uniformly $t = 0.5D$ provided the amount of paste (water and cement) in the mix is in the right amounts to hold the aggregates in position. For this reason an estimation of the water required for

workability becomes an integral part of mix design when coupled with such a grading concept.

CONTRAST OF A "WEYMOUTH TYPE CURVE" WITH A FULLER AND THOMPSON MAXIMUM DENSITY OR IDEAL CURVE OF THE STRAIGHT LINE TYPE

Grading schemes such as that of Fuller and Thompson's "ideal curve"[6] have had tremendous influence upon the establishment of coarse aggregate gradings. This use of the Weymouth formula approaches the problem from a different angle and yields different results. It is important that the results of these two procedures be contrasted.

Fuller's ideal curve was developed because it was thought that *some one grading of an aggregate of a given maximum size* would be the best gradation for use in concrete made with that material regardless of cement content. The fact that his grading produced maximum density of the aggregate alone with no paste led many engineers to specify his straight line relationship as the best gradation. As stated before, Weymouth took as his criterion the arrangement of the aggregate when in place *in the concrete*, held in position by the paste, saying that in that situation no "particle interference" should exist. Sharp differences result from the use of the two concepts.

The first contrast is that the Weymouth formula applied as in Tables 1 and 2 will distribute the material with relatively more of the intermediate sizes.* This may be shown as in Fig. 11, 12 and 13. It does not mean that less coarse aggregate *b* will be permitted but that the distribution among the sizes will be different. As an example, coarse aggregate of a maximum size of 1½ in. graded as shown in Table 2 contrasts with a Fuller gradation as follows:

Sizes	Percent by weight of each size as ideal	
	Weymouth*	Fuller
1½—¾	40.4	57
¾—⅜	32.8	29
⅜—No. 4	26.9	14

The second important difference is that such methods as Fuller's imply that some aggregate grading is ideal for all concrete of a given maximum size. This use of the Weymouth formula, along with the Talbot idea of keeping *b* constant, may be made to show that no one curve will fit all concrete made with one maximum size; in the situation shown in Fig. 11, 12, and 13, the gradation suitable for a rich

*The Furnas-Anderegg relations for maximum density [9] [10] produce distributions of the same type.

mix will be undesirable when used in a lean mix. This point is also recognized by H. N. Walsh[7].

Such use of the "spacing criterion" for good grading will result in the increased use of "pea gravel" sizes now rejected in many specifications. It will result in less "crowding the concrete full of the larger sizes," a practice of doubtful merit in many respects.

HOW WEYMOUTH'S FORMULA RATIONALIZES THE FORMULA: $p = (d/D)^n$

It has been pointed out[5] that grading curves for aggregates for many purposes should be of the general shape $p = (d/D)^n$. Tests made for this paper indicate that when the particles are spaced so that the average distance between particles, t, is less than $0.5D$ harshness results. It may be shown that a curve produced by the exponential formula with a correct exponent n and one derived by the method of Table 1 are identical. Since this is so the Weymouth formula should then be a rational method for securing the exponent n.

n for a given type of material may be derived from the following reasoning:*

Since the sieves used reduce one half in each successive sieve, then if D is the maximum size and d is the next smaller (d/D) equals $(1/2)$, and since da (see Table 1) is the amount of material in the size group larger than d, then p equals $(1 - da)$ and:

$$(1 - da) = (1/2)^n$$

The procedure then is to find da by test and to compute n.

THE DERIVATION OF n FOR TWO CONTRASTING TYPES OF AGGREGATE

	Rounded Gravel	Flat Crushed Stone
Unit weight of size group	106.2	93.2
Specific gravity	2.65	2.57
"do"	0.64	0.54
"da"	0.19	0.16
$p = 1 - da$	0.81	0.84
n, in the formula $p = (d/D)^n$	0.304	0.252

*T. C. Powers (Assistant to the Director of Research, Portland Cement Association, Chicago) submits the following proof that the two curves are identical for their entire length rather than corresponding at two points only.

The exponential grading of Fuller and Thompson is:

$$p = (d/D)^x \quad (1)$$

where p = the amount of material in the system finer than d.
d = the size of the particles in question.
D = the size of the largest particles in the system.

The Weymouth grading may be written:

$$p = (1 - da)^{n-1} \quad (2)$$

where p has the same meaning as above.
n = the number of the size group, counting the maximum size group as 1.0.

If the two curves are the same:

$$(d/D)^x = (1 - da)^{n-1} \quad (3)$$

Using the sieve series such that the openings are reduced one half in each successive sieve,

$$(d/D) = (1/2)^{n-1} \quad (4)$$

Therefore $(1/2)^{n-1} = (1 - da)^{n-1}$

solving for x,

$$x\log(1/2)^{n-1} = (n - 1)\log(1 - da)$$

and $$x = \frac{\log(1 - da)}{\log(1/2)} \quad (5)$$

Thus x depends only on da, proving the assumption to be correct.

These values correspond almost exactly with values formerly recommended for use with this formula, based on practical use of such materials.

THE NEED FOR MORE INFORMATION

To discover to what extent the relationship between ingredients in concrete can be governed by a "particle arrangement" criterion based upon calculations such as those of Table 1, a large amount of information is necessary. The author looked for it in former work. He was compelled to make many tests because of the lack of full information in available data. The primary questions which needed answers were:

(1) Is the most favorable spacing one such that t equals $0.5D$ or is it of some other order such as $.6D$, or does it vary with discoverable factors?

(2) Will a density test of one size group of a given aggregate (*do* in the formula) reflect that characteristic of that aggregate which governs its limiting amounts and water requirements for a given workability?

(3) Does the fine aggregate have sufficient influence upon the water requirement of a concrete mix to necessitate separate testing for every fine aggregate, or is there some one relationship such as the *do* value for the coarse aggregate only, which is a satisfactory starting point for mix design?

TESTS MADE

The laboratory tests made for this paper may be summed up as, *a study of the factors affecting the water requirements of concrete all brought to the same workability.* This water requirement per unit volume of concrete, w, is a direct measure for comparing such concrete mixes; viz.: if two mixes have the same quality when they have the same ratio of cement to water, it becomes evident that a concrete containing w equal to 0.20 will require cement of 0.10 at c/w equals 0.50, while one requiring w equals 0.18 will require cement of only 0.09 to be at the same ratio of cement to water. When this fact is recognized the laboratory problem then becomes one of bringing mixes to the same "workability" and comparing them on the basis of the water required for it. The reason so few data from former studies could be used for this paper was that no measures of workability other than slump tests had been made for them and because most of the data could not be reduced accurately to absolute volumes. The figures presented here are severely generalized. They present the trends taken from a large mass of data rather than precise curves of them.

Materials and methods

(1) Workability measurements. Since "workability" plays such a strong part in this study, careful selection was made of methods for measuring it. After a comparison of methods, the author decided to accumulate his data in these terms: *The same degree of workability (by interpolation from actual test data), a workability of thirty jigs on the flow table using the Remolding Effort test described by T. C. Powers*[8]. This cannot be made to correspond to any given slump and may be from two to five inches. Column 12, Table 7 is the essential information which he attempted to secure in all tests.

(2) Cement used. Only one type of cement was used. This was a standard portland cement of average present-day characteristics.

(3) Fine aggregate. For most of the tests a river sand was used. This was a typical silica sand with 80 per cent of its particles well rounded and the remainder flat or elongated. Its grading was its most unusual feature. It may be said to be a "coarse sand." See Table 3 for its grading. However, this coarseness was due almost entirely to the amount of material between the No. 8 and No. 4 sieves. Such a grading was chosen because of the fact that when Weymouth's curve is followed to the No. 8 size, exactly such a result is produced. (See Fig. 5.) Discussion is given this point under the effect of fine aggregate.

TABLE 3—GRADING OF SANDS USED FOR DATA PLOTTED IN FIG. 6, 7, AND 8

Sieve size	Fine Sand (Fig. 6), Per Cent Coarser	Sand for Main Series, Per Cent Coarser	Coarse Sand (Fig. 6), Per Cent Coarser
4	0.0	0.0	0.0
8	12.0	23.0	51.0
14	32.0	45.0	85.0
28	55.0	69.0	88.0
50	92.0	90.0	95.0
100	96.0	97.0	91.0
Pan	100.0	100.0	100.0
Fineness modulus	2.87	3.24	4.10

(4) Coarse aggregate. Gravel and crushed limestone aggregates were carefully screened to size groups and recombined for the batches in which they were used.

(a) *Specific gravity values*

Careful study was made of the specific gravity and absorption of all aggregates in order that the exact amount of materials in each batch be known. The specific gravity of the limestone of all sizes varied little from 2.57. The gravel varied greatly among the size groups. As is true of most midwest gravels, the larger particles had a higher specific gravity than the smaller. Table 4 presents these data.

TABLE 4—SPECIFIC GRAVITY AND ABSORPTION OF AGGREGATES USED

Size group	Gravel		Limestone	
	Specific gravity*	Absorption, Per Cent	Specific Gravity*	Absorption, Per Cent
1½"—¾"	2.69	1.0	2.57	2.2
¾"—⅜"	2.65	1.3	2.57	2.2
⅜"—4"	2.63	1.5	2.57	2.5

*Saturated, surface dry basis.

To prevent rate of absorption from influencing workability, the aggregate for each batch of concrete was weighed and water added 24 hours before mixing.

(b) *Study of the effect of particle shape*

Aggregates have long been considered as being of two types, crushed stone and gravel, being composed of angular or rounded particles. Such a division was contemplated at the outset of this study and supplies of each type were secured. Contradictions appeared in early test data. Inspection of these materials indicated that the crushed stone contained both cubical and flat particles and that the gravel was not all well rounded but contained many crushed particles of angular shape. It was decided to separate these materials into definite types for subsequent tests so that the effect of particle shape could be studied more definitely.

Apparatus for separating aggregates into types

For this classification the device shown in Fig. 3 and 4 was built. The operation of this equipment is based on the simple idea that particles of a given "roundness" will roll down a given incline, while "flat" ones will not.

Two factors in the operation of the separator influenced the severity of the division into flats and rounds, the slope at which the inclined belt was maintained and the speed at which it moved. These were kept constant throughout this study. The performance of the device was satisfactory. With it large quantities were separated in a short time with an efficiency of ninety per cent as judged by inspection.

A definition of just exactly what is meant by a "round" particle as contrasted to a "flat" one is not intended by the use made of this apparatus up to this point; this must be left to future developments. This author does feel that this method lends itself to becoming a test methodfor such a classification of aggregates into types as well as to a wide range of commercial applications.

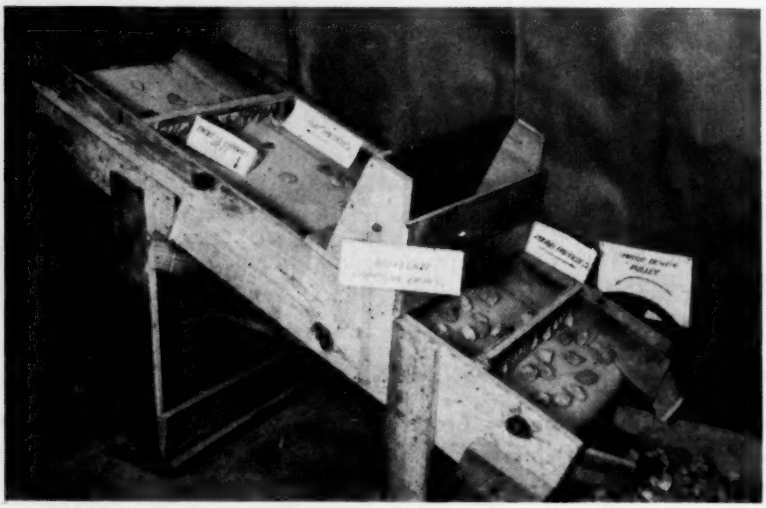


FIG. 3—AGGREGATE SORTING DEVICE

The apparatus used in the Laboratory of the Engineering Experiment Station of Iowa State College to separate aggregates into the types' "flat" and "rounded."

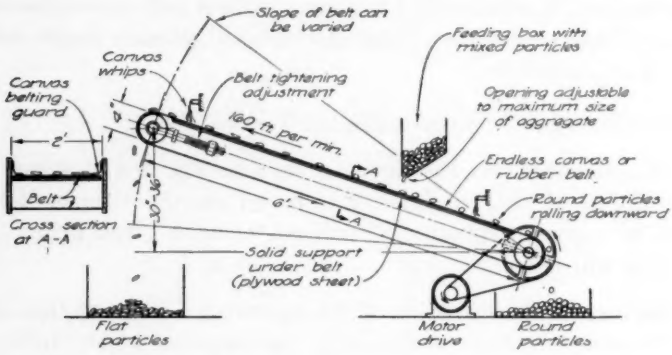


FIG. 4—DETAILS OF AGGREGATE SORTING DEVICE

A modification of this apparatus has been found useful to remove flat particles from aggregates to be used for cast stone products.

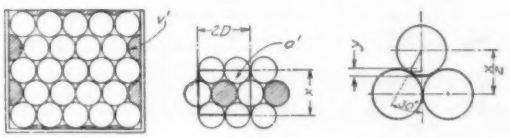
The Density "do" of Aggregates When Separated into Types and into Size Groups

Having obtained a supply of the various types of flat and round and having separated them into size groups, their density "do" as types and in combinations with opposite types was investigated. Unit weight tests were made for all sizes in a one-half cubic foot container

filled in one third increments, placed on a flow table and given ten one-half inch drops of the table for each increment. A tabular summary of some of these tests is given in Table 5.

The computation method detailed in Table 1 depends somewhat upon the assumption that *do* will be the same for all size groups of a given material. For this reason it was important to study the uniformity of this value between size groups. The results shown in Table 5 indicate that these values are not identical for all size groups; apparently increasing as the particle size decreases. This apparent change in values with size of particle can be traced to boundary conditions between the particles and the sides of the container used. That when the same container is used uncorrected test results should show *do* to decrease with increase in particle size may be proven mathematically; the steps in such a demonstration provide a picture from which a correction factor to be added to the test values may be derived*. A formula thus derived is incorrect to the extent that it assumes all boundary voids to be of the same size when actually those at the bottom and top are smaller than those at the sides. The application of this correction to the test results of Table 5 increased the values in the large sizes slightly more than would be necessary to keep *do* constant for all sizes. This situation makes it necessary to say that *do* "may be considered" to be constant for all sizes of the same material. For the purposes of this paper the *do* value determined with the No. 4 to ⅜-in. sizes was used since in that case the boundary effect is least.

*Derivation of correction factor to be added to test value for "*do*"—



SECTION SIDE VIEW

DERIVATION OF VALUE FOR x

Ideal arrangement of particles considered as spheres

$$\frac{x}{2} = D - y, \quad x = 2D - 2y$$

$$\frac{y}{2} = \frac{D}{2} - 0.433D, \quad y = 0.134D$$

$$x = 2D - 0.268D$$

$$x = 1.732D$$

Shaded portions are unfilled void spaces at container boundaries.

Each area that includes one unfilled void space equals $(2D)(x) \quad a' = (2D)(1.732D) = 3.462D^2$.

The volume of each particle v' per area a', equals $v' = \frac{\pi D^3}{12}$

For the entire container the correction will be $K = \left(\frac{A}{3.464D^2}\right)\left(\frac{\pi D^3}{12}\right)/V = \frac{AD}{13.231V}$

where A = the inside area of the container, including top.
D = the average diameter of the size group tested.
V = the volume of the container.

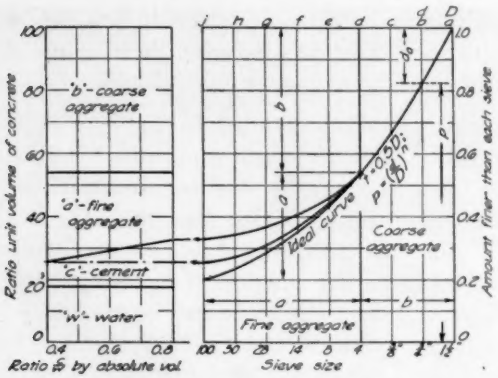


FIG. 5—IDEALIZED RELATIONSHIP BETWEEN ALL INGREDIENTS OF A RANGE OF CONCRETE MIXES PRODUCED BY THE METHODS OF THIS PAPER

This figure shows (right) (a) An ideal aggregate grading in which "*d*" equals 0.5*D*. The "rotation" of the fine aggregate about a point on the No. 4 size when "b" is held constant.
(Left) (b) The interchange of sand and cement brought about by the assumption of constant "*b*" and constant "*w*."

As shown in Table 5 the density of combinations of flats and rounds varies directly with the relative amount of each material present in the combination. Since most materials are mixtures of types rather than uniformly round or flat, it is possible that the factors influencing the use of aggregates may be studied in terms of "ideally" rounded

TABLE 5—COMPARATIVE DENSITY "DO" OF SIZE GROUPS WITH VARIOUS PERCENTAGES OF EACH SHAPE

Gravel				
% Round	0	50	67	100
% Flat	100	50	33	0
Size group				
4"—3/8"	101.5	101.8	103.2	106.2
"do"	0.613	0.62	0.628	0.646
3/8"—3/4"	97.6	101.8	103.2	106.0
"do"	0.59	0.615	0.625	0.64
3/4"—1 1/2"	97.2	101.2	103.0	104.2
"do"	0.582	0.606	0.616	0.624
Limestone				
% Round	0	50		100
% Flat	100	50		0
Size group				
4"—3/8"	89.5	90.7		91.7
"do"	0.56	0.565		0.574
3/8"—3/4"	88.2	91.0		93.9
"do"	0.55	0.567		0.585
3/4"—1 1/2"	87.7	89.9		92.0
"do"	0.546	0.56		0.573

or flat particles and interpolations made in terms of the percentages found present by test. More complete correlation of these values with the mixed concrete than is done in this paper is necessary. The author believes that the concrete making characteristics of aggregates will vary with the value *do*.

Discussion of test results

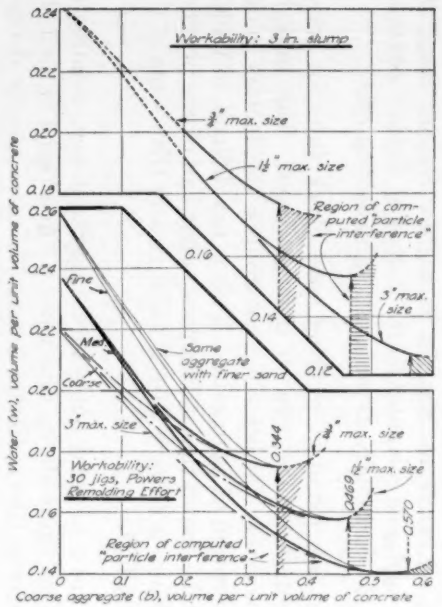
(a) *The agreement between computed optimum "b" values and test values.* The chief purpose of tests was to discover for given aggregates at what coarse aggregate content *b* the water requirement was a minimum. In Fig. 6, 7, and 8 this is the lowest point in the curves. Should this point correspond with the point at which "particle interference" begins in the coarse aggregate when computed by the method of Table 1, then the computation of *b* by such a method will be substantiated as a reliable basis for a mix design.

The essential testing procedure was to mix a batch of mortar and to add to its increments of coarse aggregate, making workability measurements with each addition. This was done with aggregates graded according to the percentages shown in Table 2 and was done for several maximum sizes with the four types of aggregate studied. The data taken are presented in Table 9. Fig. 6, 7, and 8 are derived from these data. These figures present the data more adequately than can be done by discussion. The following summary may be made from the figures:

Plotted test results indicate a *satisfactory agreement between computed optimum "b" values and the tests for round gravel aggregates of all sizes.* Some disagreement is evident; the most important conflict being *that for 1½-in. maximum size flat or angular crushed stone, the amount of coarse aggregate computed by the method of Table 1 was more than is desirable for workability as measured by remolding effort.*

Some difficulty was found in securing good check tests for water requirement *w* between mixes run on different days. This may be attributed to laboratory facilities for controlling the factors which influence the absorption of aggregates. This is not difficult to understand when it can be shown that a difference of one half of one per cent absorption may cause *w* to vary 0.01, a value greater than some of the variables being studied.

Fig. 6 may be used to point out several important facts relative to sand grading. This figure shows that when a "fine" sand is used the point of least water *w* (the optimum *b* value) is more clearly defined



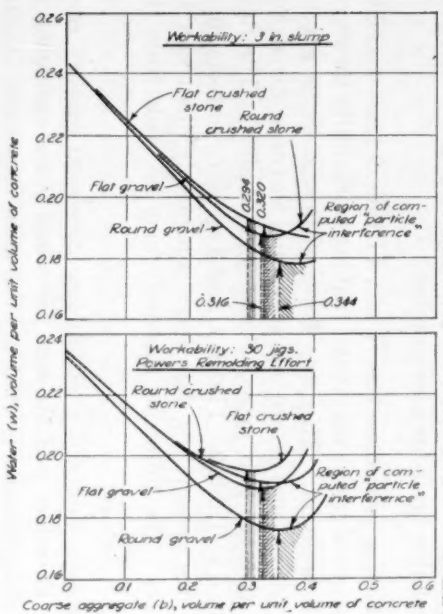


FIG. 6 (left)—RELATION OF THE WATER REQUIRED FOR A GIVEN WORKABILITY TO THE AMOUNT OF COARSE AGGREGATE IN A UNIT VOLUME OF CONCRETE

Curves derived from test results using well rounded gravel.

(a) *Upper figure.* Showing that when slump is used as a measure of workability the particle interference criterion at $t = 5D$ does not give the limiting value for "b."

(b) *Lower figure.* Showing that when Remolding Effort is used the computed value at which particle interference begins is a reasonable limiting value for "b."

(NOTE: See Table 6 for the computation method used to arrive at the "b" values for each maximum size which indicate the beginning of break to particle interference.)

FIG. 7—RELATION BETWEEN THE WATER REQUIRED FOR A GIVEN WORKABILITY AND THE AMOUNT OF COARSE AGGREGATE "B" OF A GIVEN TYPE IN A UNIT VOLUME OF CONCRETE

Curves derived from test results using ¾-inch maximum sizes.

(a) *Upper figure.* Showing that when slump is used as a measure of workability the particle interference criterion does not give a limiting value for "b."

(b) *Lower figure.* Showing that when Remolding Effort is used the computed value at which $t = 0.5D$ approaches a limiting value.

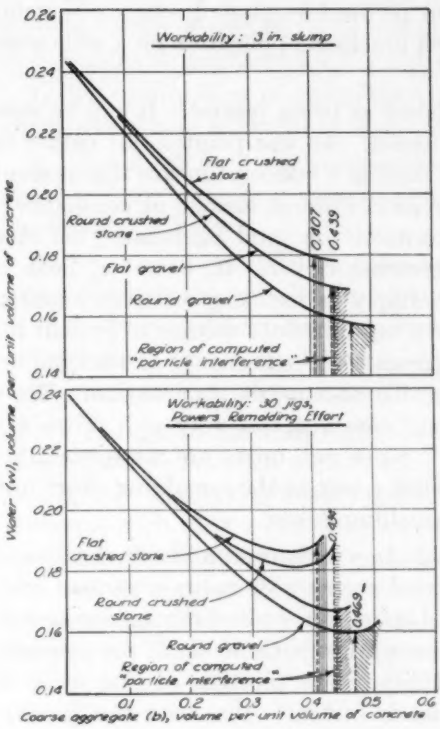


FIG. 8—RELATION BETWEEN THE WATER REQUIRED FOR A GIVEN WORKABILITY AND THE AMOUNT OF COARSE AGGREGATE "b" OF A GIVEN TYPE IN A UNIT VOLUME OF CONCRETE

Curves derived from test results using 1½ inch maximum sizes.

(a) *Upper figure.* Showing that when slump is used as a measure of workability the particle interference criterion produces limiting values for "b" with all four types of aggregate.

(b) *Lower figure.* Showing that with well rounded gravel Weymouth's formula provides a limiting value. With poorly shaped aggregates "harshness" is produced before this value is reached.

than for better graded sands, in other words the curves of Fig. 6 are more nearly horizontal for a well graded sand. This means that when a sand is well graded no great importance need be attached to discovering the exact optimum coarse aggregate, while with a poorly graded sand it may be important. It should likewise be noted that curves showing slump versus w do not indicate such facts so clearly; this is due to the fact that slump does not measure workability in the same sense as does remolding effort. With these indications in mind it seems important that distinction be made as to just what constitutes a "well graded" sand as compared to a "poorly graded" sand. These tests indicate that a well graded sand is one with the correct amounts of material between the No. 4 and No. 14 sieves as well as the smaller sizes, or according to Fig. 5 a sand in which Weymouth's criterion for particle spacing is carried continuously from the coarse aggregate into the sand sizes.

(b) *The validity of the Slater-Lyse Rule.* Many observers agree to the reasonableness of the assumption, "the consistency of concrete mixes remains constant, regardless of richness, if the gradation of the

aggregates and the water content per unit volume of concrete remains constant." This rule has reached practical application on a wide scale almost by common consent.

Such a rule cannot be established as being precise. It can be substantiated as being practically useful. As was pointed out earlier in this paper Fig. 1b indicates that holding w constant means that cement may be interchanged with aggregate without change in consistency. If used to mean exchange of cement for total aggregate, the rule becomes less true as the mix becomes richer. If, however, both w and b are held constant, then we imply an exchange between c and a, or the rule means an exchange of a unit absolute volume of cement for a unit absolute volume of fine aggregate without change of workability. The term "consistency" in the rule also needs clarification. Tests indicate that as the water is held constant and very rich mixes are used, the mixes become "drier." Since rich mixes are more workable with less slump than lean ones, such a test as the remolding effort test should show different results than slump tests.

In consequence of the foregoing observations an additional series of tests was made. These tests carried a very wide range of cement content from very lean mixes up to a point where all of the aggregate was b, the coarse aggregate. In all cases (even with no sand) the concrete appeared of satisfactory workability. The plotting of this series is shown in Fig. 14. This figure clearly indicates that over the "practical range" the water required for a constant workability is only approximately constant with b held constant. Both rich and lean mixes require more water than those of intermediate richness. Two generalities are observed: (a) In the rich mixes additional water was necessary to compensate for the large amount of cement and (b) the least water was required by mixes of intermediate cement contents. Such conclusions as these may be used only as generalities since the magnitude of such deviations will vary with such factors as the water requirements of the particular cement in use and the fineness of the sand.

This author's mix design method uses the Slater-Lyse rule modified to conform with a statement such as the following: *The workability of concrete mixes using a given aggregate will remain approximately constant over a practical range of cement contents if both the water and coarse aggregate per unit volume of concrete are held constant.*

SUMMARY AND CONCLUSIONS

This paper contains a method for preliminary mix approximation and field adjustment which grew out of the principles of several authors.

It is not intended as a replacement for the "trial batch" or any other method of mix design but as a supplement to them. Its use will furnish reasonable solutions for three situations.

1. The estimation of preliminary mixes prior to trial batches for conditions where workability is an important factor.

2. The need for "adjusting mixes" from one cement content to another if the water requirement and the coarse aggregate content for workability is known.

3. The need to establish a criterion for good grading so that aggregate combinations may be modified to remedy defective gradings or to make best use of available aggregates in the interest of workability or economy.

The laboratory tests made seem to warrant the following conclusions:

(1) The use of ideal aggregate grading curves based on a relationship between sizes such that t equals $0.5D$ is a good spacing criterion. In Fig. 6, 7, and 8, it is apparent that when coarse aggregate is used in amounts such that t becomes less than $0.5D$, more water becomes necessary to maintain equal workability. Although the test data are not entirely conclusive, these figures do indicate that, for the type of workability indicated by the remolding effort test, b computed by the method of Table 1 is a limiting amount; if more is used harshness results, if less is used more cement may be necessary to maintain the strength.

It should be carefully noted that the amount of coarse aggregate b determined by these methods is a maximum for workability of the type defined by the remolding effort test. For pavement slabs more may be used; for structural concrete it may be wise to use less.

The entire procedure for designing mixes which is suggested can be stated in a rule:

Using tests and computations such as those in Tables 1 and 6 *compute the aggregate as though it were graded from the maximum size down to the No. 4 sieve with an assigned degree of particle freedom* (t equals $0.5D$ for usual conditions) *and use this value as "b" the coarse aggregate,* (or approximate b from a chart such as Fig. 9). *Consider "b" constant for all cement contents with that aggregate. Estimate the water requirement "w" from such data as Fig. 10* (or determine it from a trial batch) *and consider "w" constant for all cement contents. Compute "c" from the cement-water ratio c/w and consider the remainder of the mix to be fine aggregate.*

This work indicates the need for more information in the following directions:

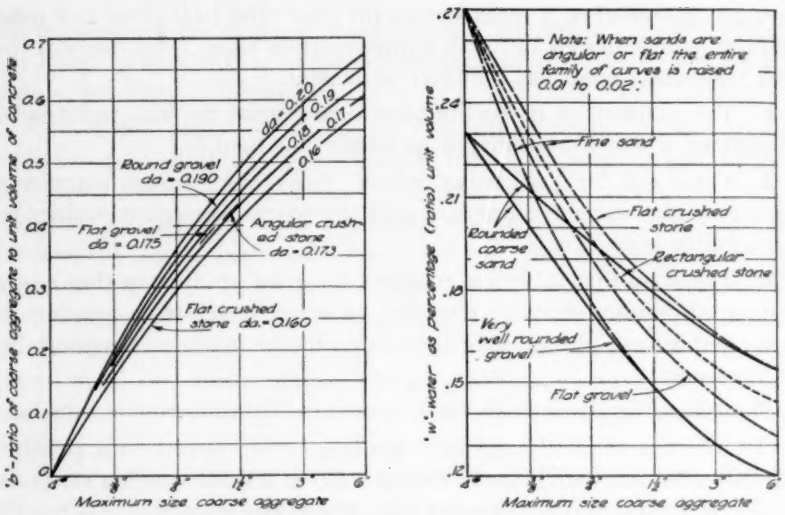


FIG. 9—RELATIONSHIP BETWEEN THE LIMITING AMOUNT OF COARSE AGGREGATE "b", THE MAXIMUM SIZE OF THE COARSE AGGREGATE AND THE TYPE OF AGGREGATE

NOTE: This is a graphic method for locating "*b*." It may be computed as is done in Table 6.

FIG. 10—RELATIONSHIP BETWEEN "w" THE WATER REQUIRED FOR REMOLDING EFFORT EQUALS 30 (A MEDIUM CONSISTENCY) AND THE MAXIMUM SIZE OF THE COARSE AGGREGATE WHEN "b" IS AT THE LIMITING AMOUNT

This figure was derived from tests made for this paper and checked, particularly in the larger sizes, by data assembled from other sources.

(1) The variations that can exist in "grading envelopes" between the maximum size and the No. 4 size and still promote workability. Present day approved gradings lie between two limits; (a) a coarser limit, most frequently used, derived from the straight line or "maximum density" relationship of Fuller and Thompson; (b) a finer limit similar to that produced by following the logic of C. A. G. Weymouth as interpreted in Tables 1 and 2. The Weymouth method like that of Furnas and Anderegg encourages the use of more intermediate aggregate sizes. The data of this paper are not sufficient to furnish direct evidence in this direction but do point out that excellent results may be obtained by following such a procedure.

(2) More accurate estimates for *w*. According to these tests water requirements seem to be influenced greatly by the type and shape of aggregate. The amount of water per unit volume of concrete required to bring any assembly of solids to a given workability has always been the important "unknown"; the essential purpose of most methods of

TABLE 6—A COMPARISON OF THE RESULTS OBTAINED WHEN THE METHODS OF THIS PAPER ARE APPLIED TO FOUR AGGREGATE TYPES

6a—Necessary Tests and Computations

		Rounded Gravel	Flat Gravel	Rectangular Crushed Stone	Flat Crushed Stone
A. *Necessary Tests*	(1)	Unit Weight 106.2	98.0	93.2	87.0
	(2)	Spec. Gr. 2.65	2.65	2.57	2.57
	(3)	Classification 100% Round	100% Flat	100% Round	100% Flat
B. *Computations*	(1)	"*do*" = 106.2 ÷ 2.65 × 62.5 = 0.64	0.59	0.58	0.54
	(2)	"*da*" = .64/3.375 = 0.190	0.175	0.173	0.160
	(3)	Space to be occupied by each size group if spacing is to be "t" equals 0.50D			

Size Group	Space available (c)	(da)	"b" Sum (c)		Space available (c)	(da)	"b" Sum (c)								Max. Size, Col. 4 = "b"
	(1)	(2)	(3)	(4)	(1)	(2)	(3)	(4)	(1)	(2)	(3)	(4)	(1)	(2)	(3)	(4)	
a-b	.190	1.000	.190	.190	.175	1.000	.175	.175	.173	1.000	.173	.173	.160	1.000	.160	.160	¾
b-c	.154	.810	.190	.344	.145	.823	.175	.320	.143	.827	.173	.316	.134	.840	.160	.294	¾
c-d	.125	.656	.190	.469	.119	.680	.175	.439	.118	.684	.173	.434	.113	.706	.160	.407	1½
d-e	.101	.531	.190	.570	.098	.561	.175	.537	.098	.566	.173	.532	.095	.593	.160	.502	3
e-f	.082	.430	.190	.652	.081	.463	.175	.618	.081	.468	.173	.613	.080	.498	.160	.582	6
f-g	Computations to this point will include all information necessary for aggregates up to 6 inches																

6b—Mix Designs

These mixes are here computed as absolute volumes per unit volume of concrete. The data may be converted to pounds per cubic foot or cubic yard of concrete by the method of Table 8.

Only two mixes are computed for each condition, a very lean mix at c/w equals 0.40 and a very rich one at c/w equals 0.80, the boundaries of the practical range of mixes.

Method computing each mix:

"w" is taken from Figure 10.
"b is taken from Table 6a.
"c" is computed as $(c/w)\ (w)$.
"a" is computed as $a = 1 - (w + c + b)$.

Max. Size	c/w	Mtl.	Ab. Vol.	Ab. Vol.	Ab. Vol.	Ab. Vol.
¾	0.40	"w"	0.175	0.185	0.190	0.195
		"c"	0.070	0.074	0.076	0.078
		"b"	0.344	0.320	0.316	0.294
		"a"	0.411	0.421	0.428	0.433
¾	0.80	"w"	0.175	0.185	0.190	0.195
		"c"	0.140	0.148	0.152	0.156
		"b"	0.344	0.320	0.316	0.294
		"a"	0.341	0.347	0.342	0.355
1½	0.40	"w"	0.150	0.163	0.170	0.180
		"c"	0.060	0.065	0.068	0.072
		"b"	0.469	0.439	0.434	0.407
		"a"	0.321	0.333	0.328	0.341
1½	0.80	"w"	0.150	0.163	0.170	0.180
		"c"	0.120	0.130	0.136	0.144
		"b"	0.469	0.439	0.434	0.407
		"a"	0.261	0.268	0.260	0.269
3	0.40	"w"	0.130	0.145	0.155	0.165
		"c"	0.052	0.058	0.062	0.066
		"b"	0.570	0.537	0.532	0.502
		"a"	0.248	0.260	0.261	0.267
3	0.80	"w"	0.130	0.145	0.155	0.165
		"c"	0.105	0.116	0.124	0.132
		"b"	0.570	0.537	0.532	0.502
		"a"	0.196	0.202	0.199	0.201

(Table 6 continued in 6c next page)

(Table 6 continued)

6c—Comparative Grading of the Total Aggregate for the Mixes of Table 6b

(The data "percent coarser by weight" is plotted to form the grading envelopes in Figs. 11, 12 and 13).

Max. Size	c/w		Rounded Gravel Ab. Vol.	Rounded Gravel % Coarser by Weight	Flat Gravel Ab. Vol.	Flat Gravel % Coarser by Weight	Rectangular Crushed Stone Ab. Vol.	Rectangular Crushed Stone % Coarser by Weight	Flat Crushed Stone Ab. Vol.	Flat Crushed Stone % Coarser by Weight
¾	0.40	"b"	0.344		0.320		0.316		0.294	
		"a"	0.411		0.421		0.428		0.433	
		b + a	0.755		0.741		0.744		0.727	
		⅜	0.190	25.0	0.175	23.6	0.173	23.1	0.160	22.0
		No. 4	0.344	45.5	0.320	43.2	0.316	42.4	0.294	40.4
		No. 100	0.755	100.0	0.741	100.0	0.744	100.0	0.727	100.0
¾	0.80	"b"	0.344		0.320		0.316		0.294	
		"a"	0.341		0.347		0.342		0.355	
		b + a	0.685		0.667		0.658		0.649	
		⅜	0.190	27.7	0.175	25.2	0.173	26.3	0.160	24.6
		No. 4	0.344	50.1	0.320	48.0	0.316	48.0	0.294	45.3
		No. 100	0.685	100.0	0.667	100.0	0.658	100.0	0.649	100.0
1½	0.40	"b"	0.469		0.439		0.434		0.407	
		"a"	0.321		0.347		0.328		0.341	
		b + a	0.790		0.786		0.762		0.748	
		¾	0.190	24.0	0.175	22.3	0.173	22.7	0.160	21.4
		⅜	0.344	43.5	0.320	40.7	0.316	41.5	0.294	39.2
		No. 4	0.469	58.0	0.439	56.0	0.434	57.0	0.407	54.4
		No. 100	0.790	100.0	0.786	100.0	0.762	100.0	0.748	100.0
1½	0.80	"b"	0.469		0.439		0.434		0.407	
		"a"	0.261		0.268		0.260		0.269	
		b + a	0.730		0.707		0.694		0.676	
		¾	0.190	26.0	0.175	24.7	0.173	25.0	0.160	23.7
		⅜	0.344	47.1	0.320	45.2	0.316	45.6	0.294	43.5
		No. 4	0.469	64.2	0.439	62.0	0.434	62.6	0.407	60.0
		No. 100	0.730	100.0	0.707	100.0	0.694	100.0	0.676	100.0
3	0.40	"b"	0.570		0.537		0.532		0.502	
		"a"	0.248		0.260		0.261		0.267	
		b + a	0.818		0.797		0.793		0.768	
		1½	0.190	23.2	0.175	22.2	0.173	21.6	0.160	20.8
		¾	0.344	42.0	0.320	40.7	0.316	40.0	0.294	38.2
		⅜	0.469	57.2	0.439	55.8	0.434	54.7	0.407	53.0
		No. 4	0.570	69.6	0.537	68.2	0.532	67.0	0.502	65.2
		No. 100	0.818	100.0	0.797	100.0	0.793	100.0	0.768	100.0
3	0.80	"b"	0.570		0.537		0.532		0.502	
		"a"	0.196		0.202		0.199		0.201	
		b + a	0.766		0.739		0.731		0.703	
		1½	0.190	24.8	0.175	23.5	0.173	23.6	0.160	22.7
		¾	0.344	44.8	0.320	43.2	0.316	43.2	0.294	41.7
		⅜	0.469	60.0	0.439	59.5	0.434	59.3	0.407	58.0
		No. 4	0.570	74.4	0.537	72.7	0.532	72.7	0.502	71.5
		No. 100	0.766	100.0	0.739	100.0	0.731	100.0	0.703	100.0

proportioning has been the estimation of this quantity. In practice it will be found to vary with so many factors that its estimation may always remain an approximation. It seems logical that a test such as that used to separate flats and rounds in this investigation should be introduced and used for the purpose of classifying aggregate so that water requirements may be more accurately anticipated.

Acknowledgements

Special acknowledgement should be given to Bill Hall, laboratory assistant for a large part in the development and use of the device for separating aggregates and for the preparation of the solution for

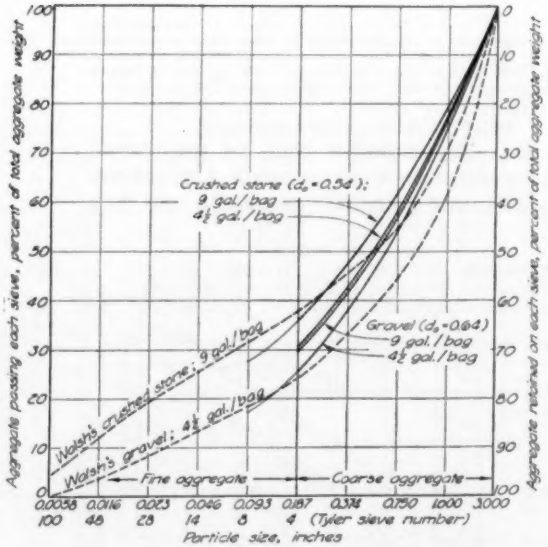

Aggregate passing each sieve, percent of total aggregate weight
Aggregate retained on each sieve, percent of total aggregate weight
Crushed stone (d_a = 0.54);
9 gal./bag
4½ gal./bag
Gravel (d_a = 0.64)
9 gal./bag
4½ gal./bag
Walsh's crushed stone; 9 gal./bag
Walsh's gravel; 4½ gal./bag
Fine aggregate
Coarse aggregate
0.0058 0.0116 0.023 0.046 0.093 0.187 0.374 0.750 1.500 3.000
100 48 28 14 8 4 (Tyler sieve number)
Particle size, inches

FIG. 11—COMPARISON BETWEEN THE COARSE AGGREGATE PORTION OF A GRADING ENVELOPE PRODUCED BY THE METHODS OF THIS PAPER AND THOSE OF H. N. WALSH

(a) Showing agreement between the amount of coarse aggregate.

(b) Showing the contrast between a typical "straight line" grading (that of Mr. Walsh) and the grading produced by the particle interference criterion.

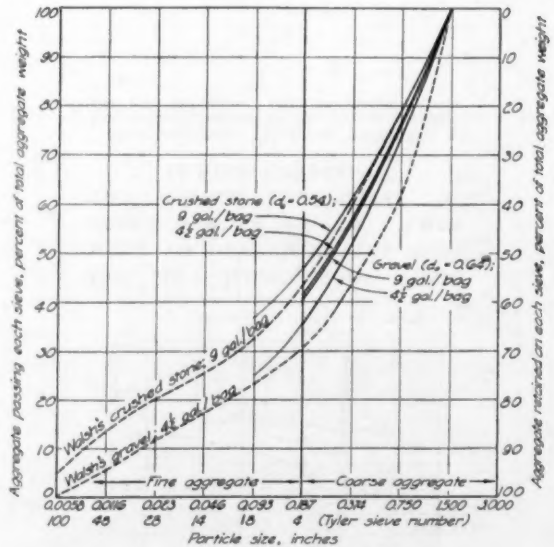

Aggregate passing each sieve, percent of total aggregate weight
Aggregate retained on each sieve, percent of total aggregate weight
Crushed stone (d_a = 0.54);
9 gal./bag
4½ gal./bag
Gravel (d_a = 0.64);
9 gal./bag
4½ gal./bag
Walsh's crushed stone; 9 gal./bag
Walsh's gravel; 4½ gal./bag
Fine aggregate
Coarse aggregate
0.0058 0.0116 0.023 0.046 0.093 0.187 0.374 0.750 1.500 3.000
100 48 28 14 8 4 (Tyler sieve number)
Particle size, inches

FIG. 12—COMPARISON BETWEEN THE COARSE AGGREGATE PORTION OF A GRADING ENVELOPE PRODUCED BY THE METHODS OF THIS PAPER AND THOSE OF H. N. WALSH

(a) Showing the larger amounts recommended by Mr. Walsh. These correspond to the amounts that would be permitted if slump tests were used as the measure of workability. (See Fig. 8.)

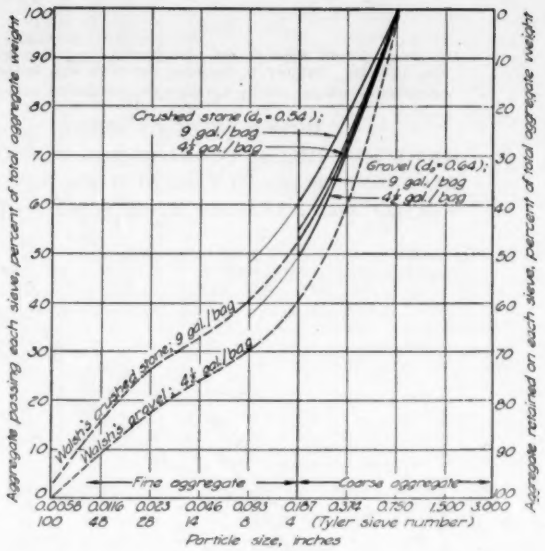

Aggregate passing each sieve, percent of total aggregate weight
Aggregate retained on each sieve, percent of total aggregate weight
Crushed stone (d_a = 0.54);
9 gal./bag
4½ gal./bag
Gravel (d_a = 0.64);
9 gal./bag
4½ gal./bag
Walsh's crushed stone; 9 gal./bag
Walsh's gravel; 4½ gal./bag
Fine aggregate
Coarse aggregate
0.0058 0.0116 0.023 0.046 0.093 0.187 0.374 0.750 1.500 3.000
100 48 28 14 8 4 (Tyler sieve number)
Particle size, inches

FIG. 13—COMPARISON BETWEEN THE COARSE AGGREGATE PORTION OF A GRADING ENVELOPE PRODUCED BY THE METHODS OF THIS PAPER AND THOSE OF H. N. WALSH

(a) Showing the much larger amounts of coarse aggregate permitted by Mr. Walsh. His amounts will be pernitted when slump is used as the measure of workability. (See Fig. 7). Trial of such concretes showed them to be satisfactory if harshness is overcome by vibration.

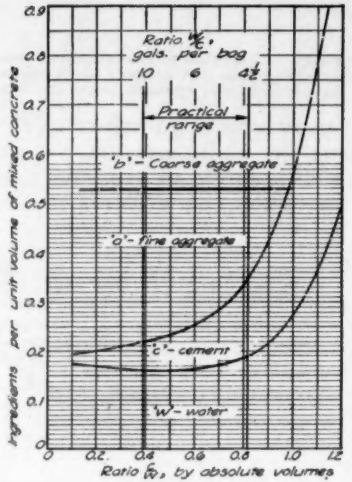


FIG. 14—RESULTS OF TESTS MADE TO DETERMINE THE CONSTANT NATURE OF "w" WHEN THE COARSE AGGREGATE "b" IS HELD CONSTANT

Test conditions: Constant workability at Remolding Effort equals 30. Coarse aggregate well rounded gravel, 1½ inch maximum size.

TABLE 7—DATA TAKEN DURING TESTS—GRAVEL COARSE AGGREGATE

TABLE 7—DATA TAKEN DURING TESTS—GRAVEL COARSE AGGREGATE

All Coarse Aggregate "b" graded according to the percentages derived in Table 2.

Batch No.	Aggregate			Ingredients by Absolute Volume				c/w	Workability Data				Compressive Strength	
									As Taken		Interpolated			
	Maximum Size	Type	Kind	b	a	c	w		Slump	R. E.	"w" at R. E. 30	"w" at 3 In. Slump	7 Days	28 Days
1. (a) Series made to determine "w" the water requirements of various mixes. Method: increments of water were added to a given mix and workabillity measurements taken with each increment.														
1a	1½	Round	Gravel	0.473	0.266	0.105	0.157	0.67	2¾	46				
b	1½	Round	Gravel	0.465	0.262	0.103	0.170	0.61	6	25	0.166	0.159	2620	4130
													3040	4170
2a	1½	Flat	Gravel	0.449	0.273	0.111	0.167	0.66	1¾	90				
b	1½	Flat	Gravel	0.442	0.269	0.109	0.180	0.60	4	75				
c	1½	Flat	Gravel	0.435	0.265	0.107	0.193	0.55	7¼	35	0.195	0.175	2300	3460
													2470	3440
9a	1½	Round	Gravel	0.399	0.350	0.094	0.158	0.60	1½	70				
b	1½	Round	Gravel	0.393	0.344	0.092	0.171	0.54	4½	32				
c	1½	Round	Gravel	0.386	0.332	0.091	0.184	0.495	8¼	12	0.171	0.163	2100	3360
													2210	3090
10a	1½	Flat	Gravel	0.376	0.344	0.106	0.175	0.60	2¼	59				
b	1½	Flat	Gravel	0.370	0.338	0.104	0.188	0.55	4½	31				
c	1½	Flat	Gravel	0.364	0.326	0.102	0.200	0.51	8½	15	0.188	0.180	2240	3586
													2200	3349
5a	¾	Round	Gravel	0.350	0.373	0.113	0.165	0.685	¾	50				
b	¾	Round	Gravel	0.344	0.367	0.111	0.178	0.624	2¾	25				
c	¾	Round	Gravel	0.339	0.362	0.109	0.191	0.570	7	12	0.175	0.179	2930	4210
													3090	4700
6a	¾	Flat	Gravel	0.337	0.373	0.121	0.170	0.710	0	150				
b	¾	Flat	Gravel	0.332	0.368	0.119	0.183	0.650	1⅝	50				
c	¾	Flat	Gravel	0.327	0.363	0.117	0.196	0.597	4	24				
d	¾	Flat	Gravel	0.322	0.358	0.115	0.209	0.550	7½	15	0.193	0.191	2730	3890
													2910	4152
13a	¾	Round	Gravel	0.418	0.324	0.099	0.173	0.571	2⅜	39				
b	¾	Round	Gravel	0.405	0.318	0.097	0.186	0.521	5½	29				
c	¾	Round	Gravel	0.400	0.314	0.096	0.199	0.482	9	10	0.184	0.177	1860	2936
													1710	2780
14a	¾	Flat	Gravel	0.388	0.331	0.106	0.175	0.605	⅞	97				
b	¾	Flat	Gravel	0.381	0.326	0.104	0.187	0.556	3¼	49				
c	¾	Flat	Gravel	0.375	0.321	0.102	0.200	0.510	7¼	35	0.205	0.187	2420	3431
													2430	3470
18a	¾	Round	Gravel	0.373	0.344	0.106	0.177	0.600	3	36	0.181	0.177		
b	¾	Round	Gravel	0.352	0.382	0.097	0.178		2½	32	0.180	0.180		
c	¾	Round	Gravel	0.320	0.411	0.091	0.180		2⅛	30	0.180	0.184		
d	¾	Round	Gravel	0.293	0.444	0.083	0.181	0.46	1½	39	0.186	0.180	1800	2490
													1730	2677
32a	¾	Round	Gravel	0.105	0.598	0.099	0.194	0.51	0	53				
b	¾	Round	Gravel	0.102	0.580	0.096	0.220	0.44	2	21				
c	¾	Round	Gravel	0.101	0.575	0.095	0.232	0.41	4½	9	0.205	0.225	1495	2380
													1550	2390
33a	¾	Round	Gravel	0.221	0.501	0.102	0.175	0.583	0	80				
b	¾	Round	Gravel	0.216	0.490	0.100	0.194	0.515	1½	20				
c	¾	Round	Gravel	0.212	0.483	0.098	0.207	0.473	4	11	0.188	0.203	2240	3410
													2300	3570

(Table 7 continued page 676)

Table 7—Continued

1. (b) Series of mixes made to check observations of series 1 (a) and to discover the relative uniformity of "w" for different cement contents. Method: The same value for "b" was used for three different c/w values for each type of aggregate.

19	1½	Round	Gravel	0.464	0.254	0.120	0.162	0.74	3¼	56	0.171	0.158	4105	4630
													4000	6040
20	1½	Round	Gravel	0.464	0.278	0.096	0.162	0.592	6	54	0.173	0.152	3000	4440
													2790	4050
21	1½	Round	Gravel	0.464	0.302	0.072	0.163	0.442	5½	57	0.173	0.152	1590	2540
													1290	2640
50	1½	Round	Gravel	0.525	0.228	0.077	0.170	0.453	5	37	0.176	0.160	1200	1830
													1340	1790
51	1½	Round	Gravel	0.525	0.202	0.102	0.173	0.590	5	85	0.208	0.160	2170	3080
													2050	3000
52	1½	Round	Gravel	0.525	0.177	0.127	0.170	0.750	4	53	0.188	0.162	3160	3860
													2600	3890
22	¾	Round	Gravel	0.413	0.290	0.128	0.172	0.745	2½	50	0.187	0.174	4450	5260
													3720	5300
23	¾	Round	Gravel	0.413	0.315	0.102	0.171	0.599	4½	55	0.189	0.166	3110	4330
													3000	4300
24	¾	Round	Gravel	0.413	0.341	0.077	0.166	0.453	2	63	0.189	0.171	1610	2480
													1660	2620
53	1½	Flat	Gravel	0.478	0.260	0.085	0.190	0.448	7	42	0.202	0.170	1410	2360
													1590	2060
54	1½	Flat	Gravel	0.472	0.232	0.114	0.183	0.623	6	63	0.206	0.170	2530	3400
													2650	3440
55	1½	Flat	Gravel	0.470	0.203	0.142	0.185	0.778	4	100	0.230	0.180	2580	3900
													3050	3310

2. (a) 200 Series. Made to locate the "optimum" "b" values more definitely. Method: To a mix composed only of mortar (sand and cement) increments of coarse aggregate were added; with each increment water was added and workability measurements taken.

201a	1½	Round	Oravel	0.000	0.615	0.143	0.243	0.69	4¼	19	0.234	0.246		
b	1½	Round	Gravel	0.327	0.400	0.093	0.179	0.52	¾	25	0.175	0.178		
c	1½	Round	Gravel	0.460	0.313	0.073	0.154	0.48	½	83				
c′	1½	Round	Gravel	0.450	0.307	0.071	0.172	0.41	4	37	0.173	0.168		
d	1½	Round	Gravel	0.471	0.290	0.067	0.175	0.38	5	40	0.182	0.168	1385	2410
													1370	2560
202a	1½	Round	Gravel	0.000	0.561	0.189	0.250	0.76	6	10	0.234	0.232		
b	1½	Round	Gravel	0.326	0.367	0.124	0.183	0.68	6¼	12	0.173	0.172		
c	1½	Round	Gravel	0.460	0.288	0.097	0.156	0.62	4	48	.....	.....		
c′	1½	Round	Gravel	0.455	0.285	0.096	0.164	0.58	5	32	0.164	0.154		
d	1½	Round	Gravel	0.480	0.270	0.091	0.160	0.57	4	55	.....	.....		
d′	1½	Round	Gravel	0.481	0.266	0.090	0.171	0.53	5¼	32	0.173	0.157	2210	3820
													2060	3500
203a	1½	Round	Gravel	0.000	0.503	0.231	0.266	0.87	5¼	12	0.249	0.252		
b	1½	Round	Gravel	0.322	0.334	0.153	0.192	0.80	5¼	14	0.180	0.182		
c	1½	Round	Gravel	0.455	0.262	0.121	0.162	0.75	4½	35	0.164	0.155		
d	1½	Round	Gravel	0.480	0.250	0.114	0.157	0.72	4	55	.....	.....		
d′	1½	Round	Gravel	0.472	0.245	0.112	0.172	0.65	5¼	30	0.171	0.153	3200	4800
													3130	4500

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204a	1½	Flat	Gravel	0.000	0.625	0.146	0.229	0.64	7½	32	0.232	0.238		
b	1½	Flat	Gravel	0.318	0.413	0.096	0.172	0.56	8½	71	.....	.....		
b′	1½	Flat	Gravel	0.314	0.406	0.095	0.186	0.51	5½	27	0.187	0.182		
c	1½	Flat	Gravel	0.396	0.356	0.083	0.175	0.47	6½	44	0.182	0.175		
d	1½	Flat	Gravel	0.440	0.317	0.074	0.172	0.43	5½	58	.....	.....		
d′	1½	Flat	Gravel	0.435	0.312	0.073	0.182	0.40	4¾	32	0.183	0.168	1465	2630
													1430	2550
205a	1½	Flat	Gravel	0.000	0.556	0.185	0.256	0.72	8	7	0.236	0.230		
b	1½	Flat	Gravel	0.313	0.372	0.124	0.191	0.65	6½	12	0.178	0.178		
c	1½	Flat	Gravel	0.387	0.328	0.110	0.175	0.63	4¼	25	0.172	0.168		
d	1½	Flat	Gravel	0.442	0.292	0.098	0.169	0.58	4¼	40	0.175	0.163	2250	4100
													2260	4000
206a	1½	Flat	Gravel	0.000	0.498	0.226	0.275	0.82	7¼	8	0.252	0.255		
b	1½	Flat	Gravel	0.308	0.336	0.154	0.203	0.76	6¼	11	0.191	0.190		
c	1½	Flat	Gravel	0.382	0.300	0.135	0.187	0.72	5	22	0.182	0.173		
d	1½	Flat	Gravel	0.440	0.260	0.121	0.180	0.67	5	50	.....	.....		
d′	1½	Flat	Gravel	0.434	0.263	0.120	0.188	0.64	5½	19	0.183	0.170	2640	4380
													2570	4340
213a	¾	Round	Gravel	0.000	0.636	0.131	0.232	0.56	1½	34	0.235	0.243		
b	¾	Round	Gravel	0.225	0.485	0.100	0.190	0.53	1	49	0.201	0.202		
c	¾	Round	Gravel	0.335	0.400	0.082	0.181	0.45	2	42	0.187	0.187		
d	¾	Round	Gravel	0.352	0.376	0.077	0.193	0.40	5	19	0.187	0.185	1450	2640
													1375	2820
214a	¾	Round	Gravel	0.000	0.593	0.175	0.231	0.76	2½	29	0.323	0.235		
b	¾	Round	Gravel	0.224	0.450	0.133	0.193	0.69	3½	16	0.184	0.191		
c	¾	Round	Gravel	0.340	0.378	0.111	0.170	0.65	2¾	28	0.169	0.172		
d	¾	Round	Gravel	0.363	0.363	0.107	0.168	0.63	2¾	31	0.169	0.170	2920	4100
													2850	4600
215a	¾	Round	Gravel	0.000	0.530	0.210	0.260	0.81	5	10	0.243	0.247		
b	¾	Round	Gravel	0.220	0.407	0.162	0.212	0.76	7	8	0.196	0.195		
c	¾	Round	Gravel	0.334	0.345	0.137	0.185	0.74	5	13	0.174	0.176		
d	¾	Round	Gravel	0.357	0.330	0.131	0.180	0.73	4½	15	0.172	0.172	3200	4500
													3680	4520
216a	¾	Flat	Gravel	0.000	0.640	0.135	0.226	0.60	1	42	0.235	0.241		
b	¾	Flat	Gravel	0.219	0.485	0.103	0.192	0.54	1½	34	0.194	0.199		
c	¾	Flat	Gravel	0.278	0.440	0.092	0.188	0.49	1¾	41	0.195	0.198		
d	¾	Flat	Gravel	0.326	0.402	0.084	0.186	0.45	2	47	.....	.....		
d′	¾	Flat	Gravel	0.322	0.397	0.083	0.197	0.42	3	23	0.196	0.197	1685	3000
													1600	3030
217a	¾	Flat	Gravel	0.000	0.580	0.175	0.245	0.71	4½	16	0.235	0.243		
b	¾	Flat	Gravel	0.225	0.446	0.135	0.205	0.66	4¾	13	0.194	0.196		
c	¾	Flat	Gravel	0.275	0.406	0.123	0.194	0.63	4¾	15	0.185	0.187		
d	¾	Flat	Gravel	0.326	0.374	0.113	0.186	0.61	3½	21	0.181	0.183	2710	4560
													2750	5000
218a	¾	Flat	Gravel	0.000	0.532	0.219	0.249	0.88	2¾	23	0.244	0.251		
b	¾	Flat	Gravel	0.216	0.341	0.169	0.205	0.82	3½	24	0.202	0.204		
c	¾	Flat	Gravel	0.276	0.376	0.155	0.194	0.80	1½	24	0.191	0.198		
d	¾	Flat	Gravel	0.328	0.347	0.142	0.184	0.77	3	31	0.185	0.184	3660	5570
													4420	....

(Table 7 continued page 678)

Table 7—Continued

CRUSHED LIMESTONE COARSE AGGREGATE

All Coarse Aggregate "b" graded according to the percentages derived in table 7.

Batch No.	Aggregate		Ingredients by Absolute Volume						Workability Data				Compressive Strength	
									As Taken		Interpolated			
	Maximum Size	Type	Kind	b	a	c	w	c/w	Slump	R. E.	"w" at R. E. 30	"w" at 3 In. Slump	7 Days	28 Days
1. (a) Series made to determine "w" the water requirements of various mixes. Method: Increments of water addde to a given mix and workability measurements taken with each increment.														
3a	1½	Flat	Limestone	0.447	0.256	0.124	0.174	0.71	½	150				
b	1½	Flat	Limestone	0.440	0.251	0.122	0.187	0.65	5	65				
c	1½	Flat	Limestone	0.433	0.248	0.120	0.201	0.60	7	67	0.206	0.184	3230	4220
													3070	4550
4a	1½	Flat	Limestone	0.420	0.264	0.130	0.186	0.70	1½	115				
b	1½	Flat	Limestone	0.413	0.260	0.128	0.200	0.64	4½	58				
c	1½	Flat	Limestone	0.408	0.256	0.126	0.213	0.59	6½	40	0.226	0.194	3260	4250
													2690	3800
7a	¾	Round	Limestone	0.324	0.352	0.133	0.191	0.70	1⅝	48				
b	¾	Round	Limestone	0.320	0.347	0.131	0.204	0.64	4	18				
c	¾	Round	Limestone	0.315	0.342	0.129	0.218	0.59	8¼	9	0.200	0.200	3510	5029
													3460	4937
8a	¾	Flat	Limestone	0.302	0.352	0.138	0.207	0.67	3¼	33				
b	¾	Flat	Limestone	0.297	0.346	0.136	0.220	0.62	7½	14	0.209	0.201	3480	5075
													3570	4775
11a	1½	Round	Limestone	0.365	0.355	0.105	0.176	0.60	1⅜	50				
b	1½	Round	Limestone	0.360	0.350	0.103	0.189	0.55	4½	29				
c	1½	Round	Limestone	0.353	0.344	0.102	0.202	0.50	8	13	0.187	0.183	2480	2954
													2530	8696
12a	1½	Flat	Limestone	0.342	0.355	0.115	0.191	0.60	2½	46				
b	1½	Flat	Limestone	0.336	0.350	0.131	0.203	0.56	6½	22				
c	1½	Flat	Limestone	0.324	0.344	0.111	0.215	0.52	9	13	0.198	0.193	2380	3830
													2510	3703
15a	¾	Round	Limestone	0.379	0.324	0.111	0.189	0.59	1½	62				
b	¾	Round	Limestone	0.372	0.318	0.109	0.201	0.54	4¼	35				
c	¾	Round	Limestone	0.366	0.312	0.107	0.213	0.50	7½	27	0.208	0.197	2760	3908
													2800	4103
16a	¾	Flat	Limestone	0.357	0.340	0.114	0.190	0.60	1½	73				
b	¾	Flat	Limestone	0.351	0.334	0.112	0.202	0.55	4½	37				
c	¾	Flat	Limestone	0.346	0.329	0.110	0.215	0.50	7	32	0.215	0.197	2720	4035

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1. (b) Series of mixes made to check observation of series 1 (a) and to discover the relative uniformity of "w" for different cement contents. Method: The same value for "b" is used for three different c/w values for each type of aggregate.

40	1½	Flat	Limestone	0.341	0.381	0.086	0.193	0.45	5	22	0.188	0.186	2640 1960	3943 3080
41	1½	Flat	Limestone	0.341	0.353	0.116	0.184	0.63	4	30	0.185	0.180	1960 3370	2940 4600
42	1½	Flat	Limestone	0.341	0.314	0.143	0.192	0.75	3⅜	31	0.193	0.189	3360 4070	4530 5300
43	1½	Round	Limestone	0.365	0.381	0.079	0.176	0.45	2½	42	0.182	0.176	4200 1990	5240 2860
44	1½	Round	Limestone	0.365	0.354	0.106	0.176	0.60	2⅜	38	0.182	0.178	1960 3200	3050 4510
45	1½	Round	Limestone	0.365	0.328	0.132	0.176	0.75	1⅜	37	0.182	0.180	3380 4750 4940	4360 5660 6200

2. (a) 200 Series. Made to locate the "optimum" "b" value more definitely. Method: To a mix composed only of mortar (sand and cement) increments of coarse aggregate were added; with each increment workability measurements were taken.

207a	1½	Round	Limestone	0.000	0.634	0.153	0.214	0.71	½	0.56	0.231	0.235		
b	1½	Round	Limestone	0.322	0.407	0.098	0.172	0.57	1	74	0.186	.....		
b′	1½	Round	Limestone	0.316	0.400	0.096	0.189	0.51	4½	21	.....	0.182		
c	1½	Round	Limestone	0.386	0.350	0.084	0.180	0.46	4	39	0.186	0.176		
d	1½	Round	Limestone	0.439	0.310	0.074	0.181	0.41	4½	46	0.190	0.173	1720 1660	2850 2700
208a	1½	Round	Limestone	0.000	0.570	0.198	0.232	0.85	1¾	26	0.228	0.246		
b	1½	Round	Limestone	0.318	0.371	0.129	0.182	0.71	4½	22	0.177	0.175		
c	1½	Round	Limestone	0.388	0.324	0.113	0.175	0.65	4¼	32	0.176	0.168		
d	1½	Round	Limestone	0.440	0.386	0.100	0.175	0.57	4¼	63	0.192	0.168	2910 2780	4450 4000
209a	1½	Round	Limestone	0.000	0.505	0.240	0.256	0.94	2½	28	0.255	0.260		
b	1½	Round	Limestone	0.314	0.335	0.159	0.193	0.82	4½	20	0.186	0.186		
c	1½	Round	Limestone	0.386	0.295	0.140	0.182	0.77	4½	42	0.187	0.175		
d	1½	Round	Limestone	0.433	0.257	0.122	0.185	0.66	5	50	0.196	0.170	3420 3450	5120 4900
210a	1½	Flat	Limestone	0.000	0.620	0.155	0.209	0.74	½	76	0.235	0.232		
b	1½	Flat	Limestone	0.310	0.400	0.100	0.180	0.55	1½	43	0.187	0.188		
c	1½	Flat	Limestone	0.380	0.356	0.086	0.178	0.48	2¼	52	0.187	0.182		
d	1½	Flat	Limestone	0.427	0.320	0.076	0.185	0.41	4½	33	0.187	0.178	1500 1600	2470 2630
211a	1½	Flat	Limestone	0.000	0.564	0.200	0.237	0.84	2½	25	0.233	0.242		
b	1½	Flat	Limestone	0.308	0.375	0.133	0.183	0.73	2½	26	0.182	0.177		
c	1½	Flat	Limestone	0.382	0.325	0.116	0.174	0.67	4	37	0.178	0.168		
d	1½	Flat	Limestone	0.432	0.290	0.103	0.174	0.59	4	58	0.182	0.167	1940 2340	4200 4040
212a	1½	Flat	Limestone	0.000	0.496	0.240	0.263	0.91	4½	14	0.249	0.252		
b	1½	Flat	Limestone	0.302	0.336	0.163	0.199	0.82	5	16	0.191	0.190		
c	1½	Flat	Limestone	0.377	0.296	0.143	0.183	0.78	4¼	52	0.190	0.177		
d	1½	Flat	Limestone	0.427	0.261	0.126	0.186	0.68	5½	49	0.197	0.175	2440 2640	4900 4230

(Table 7 concluded next page)

Table 7—Concluded

3. Series "C" made to determine the effect of the shape of fine aggregate particles upon "b" and "w."

Note: Kennedy mill sand is composed of "flat" particles. Sturtevant mill sand is same material reshaped so that particles are rounded.

	Kennedy Mill Sand													
262a	1½	Round	Gravel	0.000	0.550	0.155	0.298	0.52	3¾	17	0.290			
b	1½	Round	Gravel	0.231	0.415	0.117	0.236	0.50	4	16	0.227			
c	1½	Round	Gravel	0.374	0.335	0.094	0.187	0.48	2⅝	47	.....			
c	1½	Round	Gravel	0.366	0.329	0.093	0.212	0.44	5½	22	0.198			
d	1½	Round	Gravel	0.457	0.274	0.077	0.186	0.41	4¼	55	0.180			
	Sturtevant Mill Sand													
263a	1½	Round	Gravel	0.000	0.568	0.160	0.273	0.59	4¼	10	0.262			
b	1½	Round	Gravel	0.238	0.426	0.120	0.216	0.55	5	10	0.207			
c	1½	Round	Gravel	0.380	0.342	0.096	0.179	0.53	3½	28	0.176			
d	1½	Round	Gravel	0.475	0.284	0.080	0.161	0.50	2¼	59	.....			
d	1½	Round	Gravel	0.470	0.281	0.079	0.168	0.47	4½	51	0.190			
	Kennedy Mill Sand													
264a	1½	Flat	Limestone	0.000	0.542	0.156	0.292	0.53	4⅛	13	0.282			
b	1½	Flat	Limestone	0.232	0.417	0.118	0.231	0.51	3	25	0.231			
c	1½	Flat	Limestone	0.374	0.337	0.095	0.193	0.49	1¾	112	.....			
c	1½	Flat	Limestone	0.367	0.330	0.093	0.210	0.44	3½	70	0.203			
c	1½	Flat	Limestone	0.360	0.324	0.091	0.225	0.40	4	41	.....			
d	1½	Flat	Limestone	0.450	0.270	0.075	0.210	0.36	3½	84	.....			
d	1½	Flat	Limestone	0.437	.....	0.074	0.227	0.33	5	60	0.212			
	Sturtevant Mill Sand													
265a	1½	Flat	Limestone	0.000	0.567	0.160	0.274	0.58	3¾	12	0.257			
b	1½	Flat	Limestone	0.237	0.425	0.120	0.218	0.55	4¾	19	0.210			
c	1½	Flat	Limestone	0.374	0.337	0.095	0.193	0.49	4½	42	0.200			
d	1½	Flat	Limestone	0.461	0.276	0.078	0.186	0.43	4	133	0.257			

the density factor correction. The author is also grateful to H. J. Gilkey, F. R. McMillan, T. C. Powers, the publications committee of the Institute and others for reading the original draft of this paper and returning their constructive and candid criticisms.

APPENDIX—A CONDENSED APPLICATION OF THE PRINCIPLES OF THIS PAPER TO MIX DESIGN AND MIX ADJUSTMENT

The "design" of a concrete mix is considered to be composed of three parts:

(1) A preliminary estimate of the mix.

(2) A trial of this estimate under job conditions. This trial batch may be of any size with the aggregates in field condition. It need not be at the exact water-cement ratio to be finally used. It is made with the view that its function is to check the estimates for w and b made in the paper. From the data secured in this trial a wide range of mixes may be computed by the methods of the final step.

(3) The adjustment of the trial mix to one having the desired water-cement ratio and consistency.

At the time it is placed plastic concrete may be assumed to be a mass in which the volume (or yield) is equal to the sum of the absolute volumes of all ingredients.

Algebraically: $w + c + a + b = 1.00$ ..... (1)

where w = the water as a ratio of the unit volume.

c = the cement as a ratio of the unit volume.

a = the fine aggregate as a ratio of the unit volume.

b = the coarse aggregate as a ratio of the unit volume.

In such plastic concretes "the strength and other desirable properties are governed by the ratio of water to cement." This ratio may well be stated as a ratio of cement to water by absolute volumes c/w, such a statement assisting in the computations. Fig. 2 shows strength variations plotted in such terms with gallons per bag interpolated for practical purposes.

Since strength is determined by the ratio c/w it is apparent that w is an important value, because (a) if quality is determined by the ratio c/w and w becomes known, then c the cement content may be computed, and (b) with c and w known, then the total aggregate must equal $1 - (c + w)$.

$a + b = 1 - (c + w)$ ..... (2)

A method for estimating w and b then becomes desirable. While these can be obtained accurate'y only by "trial," they may be approximated closely by the following procedure:

(1) *The estimation of w and b*

Fig. 10 generalizes typical variations in water requirements w arising from the use of aggregates of various sizes and types. Fig. 9 presents an approximation of the maximum amount of coarse aggregate b of each size and type of coarse aggregate that may be safely used if the aggregate is reasonably well graded. (These values for b may be computed from tests as done in Table 1; Fig. 9 is a graphic method.)

The problem of selecting a trial mix is complete when these data are available. The final and most important step is that of "trying this mix," and then making adjustments if their need is apparent from the character of the concrete.

(2) *Typical computation of a mix based on these methods*

a. Given: (1) A sand with rounded particles and fairly good grading.

(2) A coarse aggregate, gravel, maximum size one inch, 50 per cent well rounded, no extreme grading condition.

b. Desired: To produce concrete with slump.....3 in.
To produce concrete with 28 day strength.....2500 p.s.i.

c. Procedure: From Fig. 2.....c/w equals 0.50
From Fig. 10..... w " 0.170
From Fig. 9..... b " 0.400
Then c equals $(0.50)w$..... c " 0.085
a equals $1 - (w + c + b)$..... a " 0.345

d. Tabulation of results:

TABLE 8—THE CONVERSION OF PARTS PER UNIT VOLUME TO POUNDS PER UNIT VOLUME OF CONCRETE

	Ratio of a Unit Vol.	x (Sp. Gr.) (62.5)	Lb. per cu. ft. of Concrete	x 27	Lb. per cu. yd. of Concrete
w =	0.170	x 62.5	= 10.6	x 27	= 286
c =	0.085	x 3.10 x 62.5	= 16.5	x 27	= 445
a =	0.345	x 2.65 x 62.5	= 57.4	x 27	= 1550
b =	0.400	x 2.65 x 62.5	= 66.5	x 27	= 1795

(3) *Mix adjustment method*

(a) When a batch from the mixer is judged to be of satisfactory workability but it is desired to alter w/c, or to change cement contents if this is the governing factor, the adjustment may be made by the use of the same assumptions as those used in the design procedure. This concept is based on the assumption that *the values "w" and "b" remain constant at a given workability over the entire range of c/w values.* (The "practical range" in Fig. 2.) Consequently a mix brought to the proper workability at any cement content or w/c may then be adjusted quickly to the desired conditions so that succeeding mixes are as desired.

(b) Procedure in adjusting a mix.

(1) The results of a trial mix.

TABLE 9

	Lb. used (gross)	Moisture	Lb. used (net)	Ab. vol. = lb. used / Sp. gr. x 62.5	Ratio to a Unit Volume of concrete
(w) water	96	....	106	1.700	0.1710 (w)
(c) cement	188	....	188	0.970	0.0975
(a) fine aggregate	565	+23	542	3.270	0.3285
(b) coarse	650	−13	663	4.010	0.4030 (b)
			Sum	9.950	

1. Workability characteristics:
Slump: 1 in. Aggregate balance satisfactory
Recommended change: Increase slump to 3 in.
2. Water-cement ratio: 6.4 gal. per bag
Recommended change: Change to 6 gal. bag

(2) The adjustment

(a) Since the foregoing mix was too dry we will apply this rule:

For each additional one inch of slump desired add 0.005 to the water "w", if too wet subtract 0.005 from "w" for each one inch reduction desired.

Using the values for w and b by unit volumes found in the trial batch correct w to give the desired slump and place this value in a table such

as Table 10 at the desired c/w and slump; put b in the table; compute c and a, then reduce all ingredients to pounds by the same method as that used in Table 8.

TABLE 10—TYPICAL FORM FOR MIX ADJUSTMENT COMPUTATIONS

showing change of trial batch to (a) three inch slump and (b) 6 gal. per bag or c/w = 0.60

c/w		.40		.50		.60		.70		.80	
W/C		.80 wt.	9.0 gal.	.645 wt.	7¼ gal.	.535 wt.	6.0 gal.	.46 wt.	5.2 gal.	.405 wt.	4½ gal.
Slump, ins.	Mtl.	Ratio unit vol.	Lb. per cu. yd.	Ratio unit vol.	Lb. per cu. yd.	Ratio unit vol.	Lb. per cu. yd.	Ratio unit vol.	Lb. per cu. yd.	Ratio unit vol.	Lb. per cu. yd.
1	*w*	.1710		.1710		.1710		.1710		.1710	
	c										
	a										
	b	.4020		.4020		.4020		.4020		.4020	
3	*w*	.1810		.1810		.1810	305	.1810		.1810	
	c					.1086	569				
	a					.3084	1379				
	b	.4020		.4020		.4020	1725	.4020		.4020	
6	*w*	.1960		.1960		.1960		.1960		.1960	
	c										
	a										
	b	.4020		.4020		.4020		.4020		.4020	

(4) *Should it be required to approximate a mix for some arbitrarily assigned conditions, the same system may be used*

As an example:

Given	*Required*
(1) Cement factor—6½ sacks per cu. yd.	(1) Water-cement ratio
(2) Use of an aggregate with maximum size 1½ in., containing about fifty per cent well rounded particles	(2) Probable strength
(3) Slump two to three inches	(3) Quantity of materials

Solution:

(6½) (94) equals 611 lb. $\dfrac{611}{(3.1)\ (62.5)\ (27)} = 0.117$ . . . . . $c = 0.117$

From Fig. 10 . . . . . $w = 0.160$

From Fig. 9 . . . . . b not more than . . . . . $= 0.450$

Then the sand a equals $1 - (c + w + b)$ . . . . . $a = 0.273$

Answers:

(1) c/w equals .117/.16 equals 0.73. (W/C equals 5.0 gal. per bag)

(2) Probable strength about 4300 p.s.i. at 28 days. (Strengths shown in Fig. 2)

(3) Quantity of materials may be computed by the method of Table 8.

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Discussion, to close in the September Supplement, should reach A. C. I. Secretary by July 20.

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(Supplement)

JOURNAL
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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *SEPTEMBER, 1940*

Discussion of a paper by W. M. Dunagan:

The Application of Some of the Newer Concepts to the Design of Concrete Mixes*

BY F. H. JACKSON, E. B. RAYBURN, JR., C. H. SCHOLER, C. A. G. WEYMOUTH AND GEORGE C. ERNST

CONVENTION DISCUSSION

Readers interested in the discussion of mix design should see discussion also of three other papers: R. F. Blanks and others; Charles T. Kennedy and Henry L. Kennedy. Convention discussion of all four papers tended to overlap.

F. H. Jackson†—(Referring to previous participation in the discussion of mix design papers): I want to correct, if possible, a possible inference that I am out of sympathy with attempts which are being made to develop these theoretical relationships in concrete mix design. I am not. Where I think we are weak is in our failure to translate the information we obtain from these studies into usable knowledge which can be used by the man on the job. I think it is for Committee 613 to do that job.

E. B. Rayburn, Jr.‡—I should like to class myself as a reactionary. I feel, as Mr. Jackson does, and some others, that the man on the job is confused by these voluminous data we throw at him. After listening this morning and this afternoon to all the various mix design theories, it brings to my mind an expression that one of my professors used to use. He said it was just like threshing old straw. We have heard the familiar names of men identified with this subject for years back. Mr. Blanks also quoted from a Roman general who lived a good many hundred years ago. It seems to me that those men gave us all the essential information; that various authors have used the information as a springboard, and taken off in different directions although they

*JOURNAL, Amer. Concrete Inst., June 1940; *Proceedings* Vol. 36, p. 649.
†Public Roads Administration, Washington, D. C.
‡Ready-Mixed Concrete Corp., Indianapolis.

are still anchored to those basic relationships. As Mr. Blanks said, the poor man on the job, if he uses the recipe, is out of luck. Mr. Blanks represents one division of the concrete design department, the large project. I don't know what volume that is in relation to the total concrete volume. Mr. Jackson represents another. He says that about 25 per cent of the total is paving concrete. There is another large field, "ordinary concrete." ranging from jobs of a hundred to two hundred yards up to 2500 yards, and that concrete usually has little or no design or supervision. You would be surprised at the number of specifications that still say 1:2:4, or call for six sacks of cement, regardless of the size of the aggregate or anything else. I wonder if we could not do something about that type of concrete.

In regard to the grading theories brought out by another one of the speakers; Mr. Blanks touched on that by saying that you still have to use the material at hand. I don't know how long it will be before material will be simplified and brought nearer to the ideal curve. Until this is done, we'll have to make trial batches on the job and use what we have.

Mr. Russell stated it was impractical to separate the sand. I know of a commercial operator who does separate the sand into three sizes. He makes no different charge for this sand. This was done in order to get a closer control of the product and to increase the fine particles passing the hundred mesh sieve. He is able to produce a sand that furnishes as high as five per cent of fine material passing the hundred mesh sieve, which is unusual for that locality. Producers are also working toward providing two or three different sizes of aggregate to be recombined. This is true of ready mixed concrete plants, of which I represent one. I do not know whether it is true in out-of-the-way places. I expect it is true of Mr. Jackson's work. In most of the states they probably divide gravel into three sizes. Where this is done it is possible to come close to the ideal grading.

*Chas. H. Scholer**—I find it difficult to decide whether I am speaking on frost resisting concrete, (referring to paper by R. B. Young), or on the design of concrete materials. These suggestions are intimately related. I was somewhat concerned over the discussion on the design of concrete mixtures when the whole trend of the discussion was related to the design for strength only with no discussion of the relation of the design of the mixture to durability. In practice where you see one project that has failed because of inadequate design as regards strength, you will see a thousand that have failed or are show-

*Binghampton, N. Y.

ing distress on account of the failure in those phases of workmanship mentioned by Mr. Young. The design of the concrete mixture bears an intimate relationship to the kind of workmanship you will secure.

While listening to Mr. Dunagan's discussion, I noticed that he suggested that it might be well to cut down on the percentage of large aggregate. I wish to express my heart felt approval of Mr. Dunagan's suggestion. I have examined many miles of concrete pavement, some of which was showing evidence of distress and deterioration. Certain characteristics stand out very vividly among the examples of unusually old concrete pavements. One of the most frequent that I have noticed is an absence of large sized coarse aggregate. To cite some examples the Ohio Post Road, some 26 years old, gravel concrete pavement in the city of Pittsburgh and a 30 year-old concrete pavement in Wayne County, Michigan, all show unusually small maximum sized coarse aggregate. This characteristic occurs too frequently to be accidental.

Although we try to avoid segregation in placing concrete, we must produce segregation at all concrete surfaces. The larger the maximum size aggregate, the more segregation we must produce. In finishing the surface and the longitudinal and transverse joints of a concrete pavement the problem becomes increasingly difficult as the size of the coarse aggregate increases. In most construction practices these difficulties are overcome by using additional water and providing excessive manipulation at these points. I believe that the same thought may well be carried over into concrete placing problems in other types of structures than pavements.

I was interested in Mr. Dunagan's comments on over-sanding. I live in a part of the country where due to a shortage of suitable coarse aggregate, we are not disturbed by a sand ratio of 90 per cent. Many structures have been built with 75 per cent to 90 per cent sand. This will make good concrete, and will get good results. There is however, more difficulty in making good concrete with such proportions than other more orthodox mixes.

I have thoroughly enjoyed the "Symphony in C Minor" that we have just heard on the design of concrete mixtures. It means much to me. I can use it in my thinking and in my practice, but many engineers cannot. One might not think to look at me that I enjoy good music, but I do. Even in music I thoroughly enjoy a "Symphony", but many of my friends more capable and intelligent than I, do not enjoy such music at all. About 90 per cent of our engineers who are out building and placing concrete get nothing out of these symphonies. They are interested in their work; they are designing con-

crete mixtures; but they will get nothing out of such a study and do nothing about it until somebody translates these symphonies into the "Oh, Johnny," swing type of music.

BY C. A. G. WEYMOUTH*

The writer notes with pleasure the growing interest in the proper spacing of particles in concrete mixtures. Professor Dunagan has treated the application of the writer's formula for particle clearance thoroughly and has developed some interesting correlations with other better known grading formulae not attempted by the writer. The author's use of the value of b is sound, since all satisfactory mixes obtained by trial or design must provide b-values giving sufficient particle clearance in the coarse aggregates.

Just what is the proper criterion for this clearance may be found, upon the study of more data, to vary somewhat from that proposed by the writer, namely $t = \frac{1}{2}D$ for continuous grading. It is likely that for three or more fully filled groups, the particle clearance should be slightly more than this, while with semi-jump gradings it can be made less by an amount depending upon the quantity of material left in the gap involved.

It should be pointed out that while Professor Dunagan recommends certain ideal gradings in the coarse aggregates, his method can be widened to include other combinaions of size-groups upon limiting the value of b for each to give proper particle clearance. (This value may be calculated by reversing the computations of Tables 1 and 2.) By entering Fig. 7 and 8 with this value, the corresponding value of w may be interpolated.

The usual ideal grading curve provides a maximum percentage for the size-group with largest diameter. The implication is that if a given maximum size, say 1½-in., is to be used, the coarse aggregate must contain the full amount complying with the ideal for that size; and if a finer mix is desired, the ideal for 1-in. or ¾-in. maximum should be selected. But a finer grading may be obtained by reducing the amount of the coarsest size without eliminating it entirely. Such conditions as spiral columns with small pitch in the hooping, walls congested with bars and conduits, wall panels where vertical placing is impracticable and the concrete must be made to flow horizontally from the ends, these offer problems that can usually be solved by reducing the amount of the coarsest size to widen the spacing of its particles and this with more economy of cement than by resorting to the next lower ideal grading curve.

*Los Angeles, Calif.

BY GEORGE C. ERNST*

This paper, by bringing together the underlying factors which affect concrete mixtures and coordinating them into a practical design method, supplies more than just another method of proportioning. For many years the fineness modulus advocates, trial batch radicals, mortar-void experts, and just plain experts, have roamed at large through the wilderness of plain concrete design and shall continue to do so, no doubt. The work of Professor Dunagan merits commendation on a basis of unbiased fundamental thinking combined with organization and application of the basic principles listed on page 650.

With the danger of becoming totally academic, the writer urges that this paper be read with the primary intent of becoming thoroughly familiar with the underlying philosophy of the author's work rather than the routine of mixture design as described in the paper. In fact, if this is done, the advocated method of design becomes the only natural one. Such a study of the paper would require, of course, several hours of mental reorientation on the part of many. From such a standpoint, the paper is an excellent starting place for any graduate student in plain concrete.

The note toward the bottom of page 653 could have been underlined, italicized, and placed in upper case letters without overemphasizing its importance. It is restated in slightly different form on page 668 in italics. In fact, after familiarity with aggregates of a given type, final adjustments may be intelligently supervised by anyone having a *proper* understanding of Fig. 1 (b) and the w/c-strength relationship. I believe that such an understanding is not at all general. The writer is also pleased to see the value of the slump test minimized, insofar as an accurate measure of workability is concerned. The tests of Table 7, which apparently were conducted to clarify and substantiate the advocated design method, also supply unusually valuable information in the field of plain concrete, particularly with regard to the effect of shape of aggregate.

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Job Problems and Practice

Some will ask questions — Some will answer them — Some will do both

A. C. I. members are invited to use this new JPP department as an informal means toward mutual help. When a problem holds possibilities of general interest the discussion will be briefed in these pages.

If you have a problem—present it; a question, ask it. If you know the answer to a question asked, or if you can contribute something which may help in the solution, or have reason to disagree with the answers or suggestions published, your contribution will be welcome.

If you know of an interesting problem whose solution has already smoothed the way of someone in the field, tell us about it—some other A. C. I. member may need just that information.

The "answers" are the answers of individuals to whom the questions are referred and not of the A. C. I. as an organization.—EDITOR

Compressed air jets as a means of placing concrete (36-54)*

A—The presentation of this subject by Mr. Davis in the January JOURNAL was at that time without accompanying comment, and as the method of placing was at such variance with accepted good practice, it seemed necessary not to let it go unchallenged. As one of the problems of placing concrete is to remove air pockets and entrained air bubbles, it did not seem logical to add (with air jets) that which you are trying to remove.

Previous job experience with a pneumatic concrete placer immediately suggested that any increase in strength might be explained by

*See January, February and April 1940 JOURNALS.

a changed W/C ratio, the air treatment reducing the amount of mixing water by evaporation before hydration took place. This conclusion was further strengthened by Mr. Davis' own curves as presented in his Fig. 2 where there is shown an increase in strength by air placing over hand placing of 75 per cent for the sloppiest mix and only 6 per cent for the driest mix shown. This could be expected as in the driest mix the air from the jets would "pipe" in fairly regular channels, while in the sloppy mix the air would more thoroughly diffuse throughout the mass and have the better chance to evaporating some of the mixing water.

The attempts of Mr. Ruettgers (Feb. JOURNAL) to duplicate the results of Mr. Davis quite thoroughly prove the fallacy of placing concrete by means of air jets.

I thoroughly agree with Mr. Ruettger's conclusion that the air jets can not properly compact the concrete. It also appears that Mr. Davis' strength curves may not make a true comparison of hand and air jet placing methods unless he takes into consideration a varying W/C ratio due to air treatment.—L. E. WRIGHT.*

What to expect of frozen concrete (36-59)

Q—Would it be safe to assume that the strength of concrete frozen after placing would merely be retarded?

A—The following comments by an A. C. I. member may be of interest. Unfortunately few test data are available which show that concrete after freezing will have only a retarded strength and will not be permanently injured. This is especially true if the inquiry refers to freezing during the initial hardening period. Apparently the rate at which the concrete is frozen and the total drop in temperature are factors in determining the amount of damage done. If definite traces of ice crystals can be found in the concrete it has probably been very definitely harmed. Crystals usually occur at the boundaries of coarse aggregate particles. Occasionally concrete structures which have been frozen have been thawed and have given satisfactory service. There appear to have been no tests to determine quantitatively the amount of damage.

Probably one of the best articles on this subject was the one by H. H. Scofield, "Some Tests to Show the Effect of Freezing on the Permeability Strength and Elasticity of Concrete," A. S. T. M.

*Arkport, N. Y.

Proceedings, Vol. 37, Part 2, p. 306, 1937. Excerpts from that paper are as follows:

Effect of Immediate Freezing:

1. Concrete frozen immediately after placing attains on the average about 50 percent of the strength of the normal unfrozen concrete at the same age. There is little difference in this respect between a 1-day and a 7-day freezing period.
2. There is some indication that older concretes have less proportionate loss of strength due to immediate freezing than those tested at earlier periods.
3. There is some indication that dry concretes suffer less injury than wet concretes, especially at early ages.
4. There is little difference between a rich mix and a lean mix in their resistance to immediate freezing.

In this connection an article by A. M. Gunzburg, "Frozen Concrete Used in Russian Buildings," in the *Engineering News-Record* for Aug. 6, 1931, is of interest. It describes a method of placing concrete in cold weather so that it freezes before chemical action begins, and the concrete is kept frozen until spring. This article also brings out the importance of soaking with water during the thawing period, as well as the importance of keeping the concrete frozen until the final thawing is started. While this is an interesting instance of the recovery of frozen concrete on a large scale, one hesitates to approve the procedure for normal usage.

There are a number of dams in the northern states built in winter weather and continuously in service for periods up to 20 years or more. Second hand information with respect to these structures which seems to indicate that the builder preferred to build dams in freezing weather. It was apparently his idea that while surfaces would be somewhat pitted and eroded in a few years following erection, the structure as a whole would be much better than if built in warm weather. A number of the structures built under this regime were examined and found to have varying degrees of surface disintegration, but the disintegration had stopped so far as could be observed and the concrete was in good sound condition. It may be that the engineer responsible for these dams was 30 years ahead of his time in getting around some of the difficulties of construction which are now being cured by the use of low-heat cement and refrigeration.

Strength of low-heat cement at 28 days (36-60)

Q—The question is, must f'_c, compressive strength of concrete, the basis of most specifications for computing working stresses in design, be taken always at the standard age of 28 days even when cements of slow rate of strength development are to be used?

The strength of low-heat concrete at the standard age of 28 days is low, yet in the following 60 days gets an increase of about 45 per cent, so that at the age of three months the strengths of low-heat and standard concretes of similar proportions are about the same.

These considerations are important in connection with construction of a dam. It is the writer's opinion that since low-heat cement is going to be used and no live load expected until several months after the construction of the slab, the strength at the age of 28 days should not be taken into account. It seems reasonable to specify a low-heat concrete that at three months will equal the strength of standard concrete of the 4,000 p.s.i. at 28 days. I should like to see discussion of this subject.

A—An Institute member points out:

In the progress report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete (1937) the specifications for quality of concrete (Sec. 301-FA) refer to the minimum allowable strength at 28 days. Elsewhere the report states that when high early strength concrete is used the 7-day strength may be taken instead of that at 28 days as a basis of design. It also implies that any ultimate strength considered suitable at 28 days for concrete made with normal cement, can be considered suitable at 7 days for concrete made with high early strength cement. The same idea might well have been extended to say: Similarly an ultimate strength at 28 days considered suitable for concrete made with normal portland cement can also be considered suitable at 3 months for concrete made with low-heat cement.

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Design of large garages

E. NEUMANN. *Beton und Eisen*, Vol. 39, No. 2, p. 17 (Jan. 20, 1940).
Reviewed by A. U. THEUER

The (German) code regulations governing the design and construction of large-sized garages are discussed. Since many of the elements governing design are not applicable to American standards (floor loadings, vehicle dimensions), there is perhaps very little information of interest for American practice.

Design of hinges and articulations in reinforced concrete structures

GEORGE C. ERNST, *Proc. Am. Soc. C. E.*, Vol. 66, No. 4, p. 589-605 Reviewed by H. J. GILKEY

Reviews past tests as an aid in explaining the action of the Mesnager and Considere types of hinge under thrust, shear and rotation. Suggests methods of design which have been found to yield satisfactory results. Includes mathematical analyses, and a rather complete list of references to the previous work in this field.

Woven wire reinforcement in silo construction

K. KOEPPEN. *Beton und Eisen*, Vol. 39, No. 1, p. 1 (Jan. 5, 1940). Reviewed by A. U. THEUER

The method of reinforcement in multiple corners or connections (i. e., where two walls cross) is discussed with some detail both in regard to the problem of proper placement and its theoretical efficiency under stress. The advantages in the use of woven steel reinforcement both in the amount of work involved and increase in speed during construction is emphasized. Observations on completed structures are stated to indicate a general absence of cracks.

Transfer of concentrated column loads to basement walls: a contribution to design

BRUNO TARTAGLIA. *Beton und Eisen*, Vol. 30, No. 2, p. 31 (Jan. 20, 1940).
Reviewed by A. U. THEUER

A simplified computation is proposed for solving the problem of transfer of concentrated column loads to basement walls. The uniformly stressed area along the footings of a wall arrived at on the basis of a spread from the base of the column at a 45° angle is replaced by an area made up of a parabola and a rectangle covering the full span between adjacent columns.

A method for the determination of carbonate in small amounts of materials

F. E. JONES, *Journal of the Society of Chemical Industry*, Jan., 1940, Vol. LIX, pp. 21-23.
Reviewed by ROY N. YOUNG

A method is described for the determination of CO_2 in building materials, such as cement, quicklime and mortar, that requires no more than 50 mg. of sample and will provide results with an accuracy of $\pm 0.1\%$. The sample is decomposed with acid and the CO_2 is absorbed in a standard solution of Baryta. The solution is then back titrated with standard acid.

Axial forces with points of application not in the center of gravity of a stressed-crossection

B. LÖSER. *Beton und Eisen*, Vol. 39, No. 2, p. 23 (Jan. 20, 1940). Reviewed by A. U. THEUER

A method is derived for determining the maximum stresses at the corners of rectangular sections in which the applied forces do not act through the center of gravity of the section. Solutions are based on two assumptions: (1) tensile forces in the tension area of the concrete active, and (2) strength of concrete in tension zero. All of the different possibilities are discussed, and approximation methods derived. Illustrative examples of the methods are included.

The first application in actual design and construction of ceiling heating coils as concrete reinforcement

H. V. SARTORIUS, *Beton und Eisen*, Vol. 38, No. 23, p. 352 (Dec. 5, 1939).
Reviewed by A. U. THEUER

In this article the structural details entering into an 8-story office building, the first in Germany to take advantage of the dual use of steel for heating conduits and reinforcement, is described in detail. Floor plans and sketches, as well as actual design calculations, are included. Special details demanded by safety regulations are stressed. A brief review of the specification set up by the responsible building inspection authorities to guard against pipe corrosion is given.

Bin walls of variable thickness

HANS GOETTLICHER. *Beton und Eisen*, Vol. 38, No. 22, Nov. 22, 1939
Reviewed by A. U. THEUER

A mathematical treatment of the problem of moment variation over the surface of the walls of a square liquid container is presented. Hydrostatic pressure distribution, uniform slab stiffness, and a full tank are assumed.

The author's conclusions based on this analysis are that the methods of elementary statics as applied to the design of this type of structure is questionable only when the ratio of height of wall to width of container equals or is less than 0.5. The use of simple statics for the case when this ratio exceeds 0.5 leads to an uneconomic design.

The effect of temperature on high-alumina cement

F. M. LEA, *Journal of the Society of Chemical Industry*, Jan. 1940, Vol. LIX, pp. 18-21.
Reviewed by ROY N. YOUNG

It has been observed that set high-alumina cement falls off in strength when cured at higher than normal temperatures. This phenomenon has been studied with pure CaO, Al_2O_3 and the results are compared with those obtained with high-alumina cement. There is strong evidence that the reduced strength resulting from higher than normal curing temperature is caused by a transformation of metastable less basic hydrated calcium aluminates into the tricalcium aluminate hexahydrate.

Heating coils as concrete slab reinforcement

OTTO GRAF. *Beton und Eisen*, Vol. 38, No. 22, p. 333, Nov. 20, 1939.
Reviewed by A. U. THEUER

Incorporating what is stated to be the first application in Germany of a hot water ceiling-heating system in the construction of two office buildings, the author here enlarges on the problems that face the designing engineer and responsible building authorities. A general description of essentials, including research, required to set up adequate safety standards are enumerated. A general survey is then presented of the research undertaken in the instance of the two buildings referred to above, and which are described in detail by the individuals who carried out the several parts of a coordinated study.

Watertight concrete pontoons for Lake Washington Bridge

Engineering News-Record, Vol. 124, No. 5, Feb. 1, 1940, pp. 50-51. Reviewed by S. J. CHAMBERLIN

After periods afloat ranging up to six months, there is no evidence of leaks through the bottoms or side walls of the concrete pontoons. The concrete was designed for maximum density with a maximum size gravel of 1½-in., six sacks of cement per yard and 5½ gallons of water per sack. An oil mixture was added at the rate of one pint per yard. The 5½-in. slump concrete was vibrated in the thin-walled sections having intricate reinforcement. Inside walls received an application of heavy black oil on removal of forms, and other surfaces were kept wet to retard shrinkage cracks.

The system calcium oxide-silica-water at 30°. The association of silicate ion in dilute alkaline solution

PAUL S. ROLLER and GUY ERVIN, JR., *Journal of the American Chemical Society*, Vol. 62, Mar., 1940, pp. 461-471. Reviewed by ROY N. YOUNG

This paper gives in detail the methods employed and observations made in a study of the system CaO–SiO_2–H_2O at 30°. The hydrated silicates are gelatinous and all adsorbed CaO. By deduction from consideration of adsorption isotherms and solubility constants their compositions were (1) $CaO.SiO_2.aq.$, (2) $3CaO.4SiO_2.aq.$ and (3) $CaO.2SiO_2.aq.$ A postulate of two sets of ions, based on an association equilibrium between $H_3SiO_4^-$ and $H_2Si_2O_6^=$, is consistent with the formulas (1) CaH_2SiO_4, (2) $Ca_3(HSi_2O_6)_2$ and (3) $CaH_2Si_2O_6$ or $Ca(HSiO_3)_2$.

The behavior of cements on treating with boiling water

F. KEIL and F. GILLE. *Zement*, Vol. 29, No. 3, Jan. 18, 1940, pp. 28-31.
Reviewed by L. T. BROWNMILLER

The experiments reported here were devised in an attempt to find an accelerated test for measuring shrinkage, as specified for the German highway cements.

The usual type of specimens were prepared. At two days the bars were put into boiling water for 9 hours. After this treatment the specimens were stored in water and over potash. Shrinkage and strengths were measured up to 21 days. The strengths of portland cements were generally decreased by the treatment in boiling water; the strengths of slag cements were hardly affected by similar treatment.

The shrinkage did not show any definite relation to concrete shrinkage so that the method is unsuitable as an accelerated test.

Plywood form panels re-used 14 times in deep bridge piers

Engineering News-Record, Vol. 124, No. 11, Mar. 14, 1940, p. 61. Reviewed by ROBT. W. BEAL

Forms for the cellular concrete piers of the Narrows Bridge at Tacoma, Washington were constructed by joining four prefabricated plywood panels at their edges to form a hollow square. Each panel (12 x 13 ft. in size) consisted of a framework

of 3 x 6's over the outside of which was glued a type of waterproof plywood + 5/8-in. thick and designed expressly for concrete forms. Thus each panel had the strength and rigidity of box-girder construction. Eight 5/8-in. tie rods to the panel sufficed to prevent spreading as the concrete was placed. All plywood surfaces were coated with a synthetic resin sealer as a protection against moisture and form oil. Details are given of the method of joining the panels at their edges in such a way as to permit easy collapse and removal. All panels were used 14 times without a single replacement.

Tremie concreting scheme simplifies foundation job

W. F. WAY, *Engineering News-Record*, Vol. 124, No. 1, Jan. 4, 1940, pp. 43-44.
Reviewed by S. J. CHAMBERLIN

A concrete armor was placed around the foundation of a bridge pier to prevent further erosion of the rock. Columns and walls were poured as deep as 27 ft. to form the outer armor and to serve as a form. This method overcame objections of heavy form stresses, horizontal joints and difficulty of anchoring a form to the steep rock. The first step involved placing and anchoring a floating template. Timber column forms containing the reinforcement were placed and the tops secured to the template. Concrete was placed by tremie in one pour on all columns excepting the longest which was placed in two lifts. Forms for the walls between columns were cut to fit the rock contour and slid into place, the columns serving as guides. On removal of inside forms the bulk of the concrete was placed in two sections.

Expansions in the Autoclave caused by magnesia

H. H. VAUGHN, *Rock Products*, Apr., 1940, Vol. 43, No. 4, pp. 48-50. Reviewed by ROY N. YOUNG

This paper deals largely with bringing about a better understanding of the effects of magnesia in cement when exposed to steam under pressure. Autoclave expansion studies were made of cements containing admixtures of pulverized fused MgO of various grain sizes and proportions. It was found that the MgO content at which high expansion in the autoclave occurred decreased as the size of the MgO grains increased. The ratio of the sectional area of the MgO grains to the area of the cement in a cross section, divided by the square root of MgO grain diameter (uniform size grains) gives the MgO content at which fissuring occurs. The expansion of MgO in steam at 300 pounds pressure corresponds with that calculated on a linear expansion of 1.3 times the original. The expansion of the cement may be considerably greater than proportional to the MgO content and accompanying fissuring of the cement. It is concluded that this excess expansion must be attributed to the action of steam on the cement grains.

Asbestos-cement and its products

HENRY R. BALDREY, *Indian Concrete Journal*, Vol. XIV, No. 2, Feb. 1940, pp. 42-45.
Reviewed by J. C. PEARSON

This is the first of a series of articles on a rapidly growing branch of the cement products industry by one who is closely associated with it. The introductory portion deals with the occurrence and production of the mineral asbestos, of which there are several forms, and of its processing into fiber. It was not until the beginning of the present century that asbestos fiber and cement were found to be capable of making boards and sheets of remarkable properties, particularly in reference to tensile strength, resiliency, toughness, and heat resistance. The manufacturing process is then outlined, which is very similar to the manufacture of paper and pulp boards, except, of course, in the manner of curing. The installment con-

cludes with an enumeration of the many favorable properties of asbestos-cement as a building material, and the many uses to which it is now put, which are to be described in detail in succeeding issues.

Concrete surfaces improved by absorptive form lining

Engineering News-Record, Vol. 124, No. 1, Jan. 4, 1940, pp. 64-66. Reviewed by S. J. CHAMBERLIN

The Bureau of Reclamation has been conducting tests to find a form lining which is effective in absorbing excess water and air to improve concrete surfaces but which will not bond to the concrete. Improved quality of the concrete is necessary to reduce erosion, from abrasion and cavitation on high dam spillways. Laboratory tests indicated that the removal of excess water and air gave better surfaces and increased the resistances to freezing and thawing and to crazing. Fabricated boards of various materials showed high absorptive qualities but bonded to the concrete. Muslin lining seemed a satisfactory method of preventing bonding. Some of the observations and conclusions from large-scale field tests at Grand Coulee Dam are: absorptive linings can be developed for mass concrete work, despite the difficult combination of factors present on a large mass concrete job; none of the boards included were entirely satisfactory; sticking complicated removal and prevented reuse; only a thin portion of the board adjacent to the concrete was effective in absorbing water; sand streaking and water coursing was eliminated; resistance to abrasion was increased; and construction procedure was not complicated.

Repairing damaged concrete on the face of Spaulding Dam

Engineering News-Record, Vol. 123, No. 25, Dec. 21, 1939, pp. 66-67. Reviewed by S. J. CHAMBERLIN

Frost action, together with the pure quality of water from melted snow, caused surface disintegration of the early concrete on the upstream face of the dam frequently exposed by varying water heights. Occasional pockets along joints had maximum depths of 4 ft. 26,000 sq. ft. of shotcrete was placed representing 25 per cent of the total upstream face area. Pneumatic gads and chipping guns, operated from barges on controlled water heights, prepared the surface. The shotcrete, applied from suspended scaffolds, for the most part was 2 to 4 in. thick reinforced with 4 x 4-in. mesh of No. 8 galvanized wire anchored in place with steel dowels. Pockets were chipped out, the spaces squared up, and holes drilled normal to the surfaces for dowels to support a grillage of reinforcing bars. The shotcrete was a 1-to-4 mix of well-graded sand ranging up to ¼-in. size, and puzzolanic cement until cold weather when a quick-hardening cement was used. After 12 hours a 1-to-5 solution of sodium silicate and water was sprayed on to combine with any liberated lime and to seal the surface for curing.

Influence of cement and aggregate on concrete expansion

THOMAS E. STANTON, JR., *Engineering News-Record*, Vol. 124, No. 5, Feb. 1, 1940, pp. 59-61. Reviewed by S. J. CHAMBERLIN

Failure of certain sections of concrete pavement in the Salinas Valley, California, due to excessive expansion, was traced to the reaction of the alkali in the cement with chert and shale in certain fine aggregate. Laboratory studies of mortar and concrete specimens indicated no excessive expansions under constantly wet, constantly dry, or alternate wetting and drying, excepting in the case of 100 per cent shale coarse aggregate. Certain specimens sealed from the air, however, showed efflorescence on exposure, with resulting cracking and crazing. One source of sand in combination with a particular cement having a sodium oxide content of 1 per cent always showed excessive expansion in a few months. The same sand with a

cement having a low sodium oxide content gave negligible expansions. Sands with small percentages of shale and chert gave negligible expansions with all cements regardless of alkali content. Accordingly, in localities having deleterious substances present in the aggregate or where the concrete may be subjected to alkali waters, present specifications limit the alkali content (sodium oxide and potassium oxide) in the cement to 0.50 per cent.

Protection against splinters and blast

ANONYMOUS. *Concrete Building & Concrete Products*, Vol. XIV, No. 12, Dec. 1939, pp. 226-228.
Reviewed by J. C. PEARSON

Concrete journals from abroad these days have much to say about air raid protection. This little but enterprising periodical is no exception to the rule, and in the above article emphasizes the economy and efficiency of using standard concrete building blocks, 18″x 9″ x 9″, as compared with sand bags for revetment purposes. The main appeal for sand bag protection was speed of erection, but now rotting bags are not only starting to destroy this protection, but are most unsightly. The article points out that a good revetment should be cheap, durable, easy to erect, stable, constructed of materials readily available in most districts, easy to demolish, and of good salvage value. Standard building blocks meet all these requirements except in first cost, and in replacements of sand bag installations, the sand for filling them is at hand. In double layers, alternate header and stretcher coarses are laid, and for moderate heights they can be laid without mortar. The total cost per square yard of such revetment is estimated as 25 s. per square yard, whereas sand bag revetment of equal protective value would run up to 48 s. per square yard. Further data on protective values are given in the Jan. 1940 issue.

Reinforced concrete in the construction of the German Auto Works at Fallersleben

K. BACH. *Beton und Eisen*, Vol. 38, Nos. 12 and 13, June 20 and July 5, 1939
Reviewed by A. U. THEUER

The layout, construction, and engineering supervision provided for a large scale integrated industrial development is described. The first unit consists of four main buildings with an aggregate floor space of 230,000 sq. yds. and subsidiary buildings with a floor space of 32,000 sq. yds. Two of the main buildings, an automobile tool-shop and body plant, are of reinforced concrete throughout; a press-shop is of reinforced concrete with steel roof and a power plant is of steel construction throughout. The work in progress is being carried out by a combination of six construction companies. Among the principal difficulties at the outset were a shortage of labor and ability to obtain materials in sufficient quantity according to a time schedule. Living accommodations were provided on the site for 8400 people; however, to date the maximum number engaged at any one time was 5300. The estimates of materials required are 35,000 metric tons of high strength steel reinforcing bars, 15,000 tons of structural steel and 260,000 metric tons of plain and reinforced concrete. Out of 52,000 metric tons of cement used during 1938, some 30,000 metric tons were imported from Italy. Some difficulty was experienced also in obtaining the lumber needed. The article is well illustrated.

Precast concrete slabs face bridge abutments

L. G. SUMNER, *Engineering News-Record*, Vol. 124, No. 1, Jan. 4, 1940, pp. 61-63.
Reviewed by S. J. CHAMBERLIN

The abutments of the Clinton Avenue bridge on Merrit Parkway are faced with thin precast concrete slabs which also served as front forms during construction. Returns and moldings cast integral with each section provided sharp clear out-

lines. The 2-in. slabs were reinforced with 3-in. mesh of ¼-in. galvanized wire, and were provided with U-shaped steel plates for handling, setting and anchoring. The desired surface appearance was obtained by the use of special aggregates and cement for only the outer 1 in. Within 12 hours after placing the concrete units were removed from the composition-lined forms and placed upright with the surface exposed for curing, surface brushing, grouting, and washing with a muriatic acid solution. At the site the slabs were set on the mortar beds containing lead pellets, form ties placed, bracing applied and the backing placed. By being made at the plant, the slabs were made of carefully controlled concrete of high quality, intricate form work at the job was avoided, appearance was controlled in advance, unsightly joints were eliminated and there was an abundance of equipment available. A minimum compressive strength of 7500 p.s.i. at 28 days, and a maximum absorption of 7 per cent were required. The price for the slab facing was $1.30 per sq. ft., and the price for cast stone coping was $3.00 per cu. ft.

Workability of concrete

K. Walz, *Beton und Eisen*, Vol. 38, Nos. 21 and 22, p. 327 (Nov. 1939).

Reviewed by A. U. Theuer

Following a general discussion of what factors relevant to practice may be included under the term workability, the author summarizes them as follows:

1. The most important factors in judging workability are the tendency to segregate and the resistance to deformation.
2. These two properties do not necessarily exhibit parallelism and so must be evaluated separately.
3. The resistance to deformation (change in shape) depends on the method used to place as well as the kind of concrete.
4. To measure resistance to change of shape an instrument must indicate the work necessary to consolidate an uncompacted mass of concrete; to measure segregation the amount of segregation of a quantity of concrete must be determined following a movement over a definite distance.

A number of workability test procedures are then described by means of test data to indicate to within just what limits singular and reliable results may be expected. The slump cone, the flow table, a special flow table deformability test, the penetration test (Graf) and the sieve on flow table tests are discussed in order. All of these tests are considered as satisfactory, within the limits generally found for field placed concrete and as simple and direct methods of control. A final paragraph discusses the desirability of measuring the energy input of concrete mixers on the job.

The possibilities of the use of acid slags

G. Muszgnug. *Zement*, Vol. 29, Nos. 4, 5, Jan. 25, Feb. 1, 1940, pp. 37-41, 51-56.

Reviewed by L. T. Brownmiller

In working the low iron ores of Germany, the question of the disposal of the high silica, acid slags has become important. The slowly cooled acid slags do not crystallize sufficiently to show appreciable cementitious properties, while the quickly cooled slags only show a small latent hardening property. The author has reported here a few experiments designed to use these slags commercially.

Some of these slags are suitable for use as a raw material for the manufacture of portland cement.

The slags also may be used as an admixture to portland cement when they show hydraulic properties. Qualitatively the amount of slag is limited to 20 to 30 per cent of the portland cement clinker. The author gives some strength data obtained on such slag cements. There are no technical difficulties in the use either for the

manufacture of portland cement clinker or for the manufacture of slag cements. But the problem of the use of the slags is an economic problem.

Under certain conditions the granulated acid slags may be used for making furnace brick. For such uses it is necessary to add cementitious materials such as cement or lime, and to use a definite method for curing. The procedure for the use of the acid slags is different from that required for basic slags in the production of brick. And again the question of the substitution of the acid slags for the basic slags becomes an economic question.

Special cements in Sweden and in large American dams

B. HELLSTRÖM, *Cement & Lime Manufacture*, Vol. XIII, No. 3, Mar. 1940, pp. 37-47.

Reviewed by J. C. PEARSON

This paper, a translation from *Teknisk Tidskrift*, Stockholm, contains a short introduction reviewing the adverse effects from heat evolved from hydration of cement in large masses of concrete, and the various methods that have been followed in the construction of large dams to minimize these effects. A large part of the paper is a summary of experience on notable dams erected in the United States in the last decade, with excellent photographs and brief descriptions of them, and the means taken to reduce temperatures. Included are Boulder (low heat and modified, with water cooling); Parker (low heat with water cooling); Marshall-Ford (low heat); Seminoe (low heat with water cooling); Bartlett (low heat); Grand Coulee (modified and low heat); Hiwassee (low heat); Shasta (low heat); Friant (low heat with pozzolana). Bonneville dam is also mentioned in reference to increasing interest in pozzolanic cements. In summary author states that the advantages of low heat cements have been clearly demonstrated, although crack formation has not been altogether excluded.

In Sweden low heat cements have been manufactured of low lime and high silicate composition. These are slow hardening, but eventually acquire strengths equal to or greater than that of normal portland. Pozzolanic types are also used, and a number of power stations have been built with them. On the other hand, the author states that ordinary portland cement has been preferred for a number of recently constructed plants.

Acid treatment of concrete pavements

Annual Report, Road Research Board, London, for year ending Mar. 31, 1939, page 126.

HIGHWAY RESEARCH ABSTRACTS

In the preceding annual report of the Road Research Board the results of several studies on the coefficient of lateral friction on concrete pavements were given. As some of these coefficients were quite low, it was deemed advisable to develop a method of increasing them. One of the methods used was the application of a dilute hydrochloric acid which etched the surface of the smooth pavements.

Applications varying from 7.5 to 75 sq. yd. per gal. of concentrated hydrochloric acid were made on a pavement that had a coefficient of 0.37 to 0.51 at 30 m.p.h. On sections treated at rates of 75, 60, and 7.5 sq. yd. per gal. the corresponding friction coefficients that resulted were 0.41 to 0.57, 0.41 to 0.58, and 0.62 to 0.67, at 30 m.p.h. respectively. There was little effect on the coefficients at 5 m.p.h.

Impressed by the success of the treatment on the experimental road, the Ministry of Transport decided to carry out further full scale experiments on a concrete road that had coefficients as low as 0.15 at 30 m.p.h. The intensity of the treatment in this case was varied from 5 to 50 sq. yd. per gal. This resulted in an increasing coefficient at 30 m.p.h. up to an intensity of treatment of about 20 to 25 sq. yd. per gal. The coefficient at this point was about 0.65, and the maximum concentra-

tion of the acid, 5 sq. yd. per gal. resulted in only a 0.70 coefficient. However, a much rougher texture resulted from the heavier treatment, which it is believed will be more durable under traffic and therefore more economical. Further skidding tests will be made to determine the extent to which this is true.

Revision in German standard specifications for portland, slag and blast-furnace cements

ANONYMOUS, *Cement & Lime Manufacture*, Vol. XII, No. 12, Dec. 1939, pp. 248-249.
Reviewed by J. C. PEARSON

The German specifications, which have been in effect since 1932, were revised as of June 20, 1939, the revisions to be in effect after June 30, 1940. The major change is substitution of a flexural test for the direct tension test; requirements for compressive strength of normal cement have also been increased. The method of testing is to be in accord with the German Standard Specification DIN 1165, "Arrangements for making and testing prisms 4x4x16 cm. of soft-mixed mortar," and DIN 1166, "Preparation and testing of prisms of soft mixed mortar." Comparison of the new with the old requirement may be made in the following table:

	All Strengths given in p.s.i.			
	1932 Requirements		1940 Requirements	
	Compression	Tension	Compression	Tension
	Normal Cement			
3 days storage in water	....	...	....	...
7 " " " "	2560	256	2845	256
28 " " " "	3911	356	4267	356
28 " combined storage*	4978	427	5689	427
	High Grade Cement			
3 days storage in water	3556	356	3556	356
7 " " " "	....	...	....	...
28 " " " "	5689	427	5689	427
28 " combined storage*	7111	569	7111	569

*1 day moist air, 6 days water, 21 days air.

Tests of heating coils used as slab reinforcement

DR. SCHREIER, *Beton und Eisen*, Vol. 38, No. 23, p. 345 (Dec. 5, 1939).
Reviewed by A. U. THEUER

A detailed description of a series of cyclic heating and cooling tests carried out on slabs under superimposed loads is here given. Three slabs, two of normal portland and one of high early strength concrete, were tested. The slabs were 16½ ft. long, 3¼ ft. wide and 10 in. thick, and were insulated around their edges. Two of the three slabs were subjected to loads of 75 lb. per sq. ft.; the first being subjected to 35 cycles of heating and cooling (50°C to room temperature) and the second to ten cycles of heating and cooling (70°C to room temperature). Cooling, in both cases, took place naturally under the lower temperatures of the surrounding air. A third slab was subjected to a load of 150 lb. per sq. ft. and given 10 cycles of heating and cooling (70°C to room temperature). Cooling in this case took place by forced circulation of cold water through the pipes. Following the heating-cooling tests under load each slab was loaded to destruction, and pieces cut out for bond tests. Temperature variations at varied depths through each slab were measured by means of thermo-couples. Deflections, total amounts of cracking, and maximum crack widths resulting from all causes were measured.

In conclusion it is stated that the deformations and deflections measured were in all cases larger for the heated than for unheated slabs, but remained well within limits set by good practice. The total amount of cracking was very nearly doubled by the heating-cooling treatments. All cracks, however, were hair cracks and the maximum width recorded was 0.006 in. Injury to the slabs by the special treatments imposed were not found in any case.

Detecting unsound chert in aggregates

CHARLES E. WUERPEL, *Engineering News-Record*, Vol. 124, No. 19, May 9, 1940, pp. 80-82.
Reviewed by S. J. CHAMBERLIN

Chert in gravel varies in color, hardness and physical appearance, and the resistance of this variable material to frost action varies from very high to very low. The lack of a practical test for differentiating between durable and non-durable chert has made the acceptance or rejection of some aggregates difficult. The relative massiveness of the concrete structure also influences the acceptability of an aggregate.

A combination of monobromobenzene and bromoform was adopted as a flotation solution for separating the particles of representative samples of aggregate. Cherty gravels were separated into four groups: −2.30; 2.30-2.40; 2.40-2.50; and +2.50. Petrograph—microscopic examinations and tests for absorption, true specific gravity, resistance to the magnesium sulphate accelerated soundness test, and resistance to frost action when confined in concrete were made on each group of material. The analysis indicated the cherts to be basically cryptocrystalline silica with varying amounts of other minerals, and to vary from oolitic structure to a uniform dense non-fossiliferous texture. All light particles (below 2.40) were extremely porous. Absorption decreased markedly as the specific gravity increased. True specific gravities were found to be practically identical, the difference in apparent specific gravities being due to pores. Loss from freezing and thawing and from the magnesium sulphate test also decreased markedly with increase in apparent specific gravity. All non-chert material which floated in the 2.30 and 2.40 solutions were definitely unsound and undesirable for use in concrete. Some unsound chert particles were in the higher gravity groups, their relatively large iron oxide content offsetting the buoyant effect of the pores. All of the equipment may be packed in a portable kit for field use.

The use of high strength steels and methods of economizing in reinforced concrete construction

ERICH FRIEDERICH, *Beton und Eisen*, Vol. 39, No. 1, p. 11 (Jan. 5, 1940).
Reviewed by A. U. THEUER

These problems peculiar to the use of high strength steels, brought to the forefront in Germany by the drive towards steel economy, are touched on with special reference to existing standard specifications for reinforced concrete. The author draws the reader's attention to the increased significance of "cracking loads." Whereas with structural grade steel, cracking loads and permissible loads are very nearly identical, with the use of high-grade steels the former lies far below the latter. This factor, long militating against the use of high strength steel, has in large measure been overcome with the successful introduction of prestressing and it is this factor the author states, which accounts largely for the many new methods and formulas being advocated for design. The author then defends the present German standards in some detail. Better methods of making and placing concrete, with the goal of reaching compressive strengths for field-mixed concrete of 14,000 p.s.i., and prestressing to compensate in part for the serious weakness in tension of concrete, rather than quibbling over design formulas, are looked upon as the solution for the impasse reached.

Following this is a classification of various points entering into design formulas following the German standards and their significance in regard to conventional concrete, the properties of steels, the relation of steel to concrete in tension and flexure, the breaking load and constant n, and limits for reinforcement. A special

paragraph is devoted to both ordinary prestressed concrete, and the new steel-wire concrete.

The determination of the course of stiffening

DR. KEIL. *Zement*, Vol. 28, No. 52, Dec. 28, 1939, pp. 729-732.

Reviewed by L. T. BROWNMILLER

In mixing and placing concrete, a stiffening of the mass is often apparent which is sometimes interpreted as a lack of water or a loss of water when compared with the original consistency. Therefore the author thought it of value to measure this water loss by measuring the amount of water required to regain the original consistency after the mass had been allowed to stand for varying intervals. Such experiments were made on neat cement, cement mortars and concrete.

Eight neat cement mixes of normal consistency were prepared and allowed to stand. The mixes were progressively reworked at intervals of 1 hour, 2 hours, and so on to eight hours. After reworking, the consistency was again measured; water was added to those mixes which did not have normal consistency after reworking and the amount required to bring back that consistency was measured. The mixes which were allowed to stand for periods up to the time of initial set required less than five cc. of water to regain normal consistency. The amount of water required for those mixes which had stood for periods longer than the time of initial set progressively increased. Thus one can determine the time of initial set from such experiments; the set occurs when the amount of water required to regain normal consistency is 1 cc. to 5 cc. by following the procedure of reworking the mix after standing.

A similar experimental procedure was carried out with cement mortars. However the flow was measured instead of the consistency. The amount of water required to bring back the original flow followed very closely the loss in flow as measured in centimeters. However the time of set could not be predicted as accurately from the mortars as it could from the neat cement pastes.

Similar experiments were performed with concretes but the data do not seem to show much regularity. From this set of experiments the author's only claim is that the stiffening of concrete during the first two hours is not due to the cement; otherwise similar stiffening should have been encountered in the mortars. But the stiffening in this case is due to the absorptive action of the aggregate.

Concrete for Pensacola Dam

M. G. FULLER, *Engineering News-Record*, Vol. 124, No. 5, Feb. 1, 1940, pp. 42-46.

Reviewed by S. J. CHAMBERLIN

Thin arches, 2 ft.-4 in. thick at the top to 4 ft.-4 in. at the bottom, 140 ft. high on 60-ft. spans supported by thin-wall buttresses 84 ft. center to center, make up much of the mile long Pensacola Dam. In addition there is a spillway gravity section and a non-overflow gravity section. Placement of the different mixes for various parts of the project called for a flexible and mobile system. From the mixer the concrete was carried in 2-yd. dump buckets by a fleet of light trucks to the various pours. For the upper parts of the thin-walled arches and buttresses the buckets were picked up by high-frame gantry whirley cranes and the concrete discharged into elephant trunk hoppers. The thin-walled sections were limited to 15-ft. lifts with 5 day intervals. Crawler cranes placed the concrete for the lower portions of the arches and foundations. On gravity sections the concrete was placed directly by bucket in 5-ft. lifts, with 3 days between vertical sections and 2 weeks between adjacent side sections. Metal identification tags accompanied each bucket through its round trip to insure the concrete reaching its proper designation. Steel-lined

wood forms for the variable lower monoliths of arches and buttresses and stee forms for all other monoliths were handled by cranes. Forms were stripped at 12 hours, excepting for the upstream faces of the buttresses and the intrados of arches which were stripped at 48 hours. Curing was by water spray excepting the underside of arches which were cured by a liquid compound.

The fine aggregate was quartz sand transported 60 miles by rail. Suitable grading was accomplished by the rejection of excessive middle-sizes and the addition of fines from local aeolian deposits. The coarse aggregate consisted of ¾-in., 1½-in., 3-in. and 6-in. limestone, crushed and prepared at the quarry 22 miles away and brought in by rail. Bottom dump cars discharged the aggregate into unloading hoppers on to a belt conveyor, to a distributing conveyor, into stock piles over a reclaiming tunnel. A sprinkling system over the live storage piles reduced dust, cooled and moistened the stone. The modified cement received in bulk was distributed to the mixing plant or silo by screw-and-bucket conveyors. All ingredients were batched by automatic weighing hoppers, the system providing for four different mix settings at one time.

Difficulties met in the application of impermeable membranes to structures subject to hydrostatic pressure

HANS KLIPPLE. *Mitt. a.d. Gebiete des Wasserbaues*, etc., Vol. 10 (1940)

Reviewed by A. U. THEUER

This is a technical report of some 30 pages detailing the structural problem involved in waterproofing foundations and sub-surface structures that are subject to hydrostatic pressure. The report proper contains the results of observations and measurements made in the field, and analyses of various causes of failure. The failures described are traced back principally to faulty structural design and not to the workmanship of placing or the materials used to secure water tight dams. A brief summary of the conclusions are as follows: The majority of failures in waterproofed membranes are traceable to the failure of the elastic properties of the bitumen under construction or service conditions. A change in state of the bitumen under pressure from a solid to a semi-fluid over large areas due to heat was actually observed. An asphalt impregnated sheeting must hence be considered as a frictionless material to be correctly used in design.

In measures to provide against sliding of structural elements, those factors which affect the viscosity of the bitumen must be considered of first importance. In such structures as foundations for chimneys, furnace pits, heating tunnels, etc., a thin concrete cover is not enough. The heat of set of concrete in massive elements can serve to change the viscosity of an asphalt dam to a dangerous point. Temperatures above 30°C are stated to be commonly attained in actual concrete elements during set. A concrete protective coating of from 2 to 3 in. is considered insufficient to prevent dangerously high temperatures from large concrete masses reaching the asphaltic dam.

The necessity of reinforcement in concrete protective coatings should be determined on the basis of static considerations, i.e., presence or absence of tension, just as in all other reinforced concrete work.

The tendency to throw all the responsibility for the efficiency of a water dam on the workmanship, materials and methods of installation, is regarded as the principal cause of most failures. (Author regards these elements, where specialists in the field were employed, above criticism.) Important failures, when they do occur, can invariably be traced to faulty design. Chief among these is the non-provision of anchorage against sliding of structural elements resting on waterproofing mem-

branes; failure to consider the effects of shrinkage; proper provision for expansion joints in a way such that the insulating materials are subjected to uniform pressure from opposite sides.

Also mentioned as causes is the undependability of the active-lateral pressure of earth fills as a means of securing tight joints between walls and abutting slabs when the depths are less than 15 ft. Author recommends that advantages be taken of passive earth pressure in design and where such cannot be obtained naturally, artificial measures to do so be provided.

As a final conclusion the author states that vibration or ramming operations in the close proximity of existing structures does not in any way endanger properly placed membranes.

Prestressed concrete tank features special details

R. C. KENNEDY, *Western Construction News*, April, 1940. Reviewed by HENRY D. DEWELL

A 3½ million gallon cylindrical reinforced concrete reservoir, 160 ft. in diameter and 24 ft. 6 in. high, recently completed in Oakland, Calif. for the East Bay Municipal Utility District, presents some unusual details.

A foundation ring, 12 in. thick by 3 ft. 6 in. wide, supports, but is not connected to, the wall and the edge of the floor slab. The concrete floor is 6 in. thick, and the sub-grade was sealed before pouring concrete by spraying with emulsified asphalt to prevent absorption of water from the wet concrete. Sand was then spread over the asphalt to avoid tackiness. The reinforcing steel in the slab is continuous, but grooves were left at construction joints and later filled with a compound of hot latex and road oil.

The reservoir wall was constructed in 18 panels which were poured alternately. These panels were keyed one to another, but no reinforcing steel crosses a vertical wall joint. In pouring adjacent panels, the edge of a poured panel was painted with a 1:3 mortar of cement and iron dust. A recess 8 in. wide and ½ in. deep was provided at the interior face of each vertical joint and was later filled with this mortar.

A 2¼ in. wide annular space was left between the thickened edge of the floor slab and the inside of the wall. This space was later filled to within 3 in. of the top with cement-sand mortar. In the remaining 3 in. deep space, a water-seal was constructed, consisting of a rectangular rubber strip 2 in. wide and 1½ in. high, compressed between the convex surfaces of two 2 in. by ½ in. half-oval steel sections. At 6 in. intervals, ½ in. bronze stud-bolts were placed through the steel and rubber. Upon tightening the nuts of these bolts by means of a limited-torque friction-wrench, the rubber was expanded against the concrete surfaces, creating a computed side pressure of 175 lbs. per lin. in. After adjustment, the steel was painted with liquid rubber paint over a primer. This detail of construction is a departure from the usual practice in prestressed tanks. Standard practice fills this annular space with a cement-iron dust mortar. Subsequent expansion and contraction have caused slight seepage in such joints.

The concrete walls were prestressed by bands of 12 ⅞-in. bars, 43 ft. long, with ends upset and threaded to receive standard 1¼-in. wrought iron nuts, 1½ in. high. These rods were inserted into drilled holes in 12 built-up steel beams, 3½ in. high by 4¾ in. wide. The rods were tightened with special jointed wrenches so designed that a 6d copper nail inserted through the handle would be cut in double shear at a predetermined torque. The rods were specified to have a minimum elastic limit of 55,000 p.s.i., and ultimate strength of 85,000 p.s.i.

Experience of the District has shown that the bands of the present method are cheaper to install, can be more accurately stressed, and have better bearing on the concrete wall than the continuous bands of round rods with turnbuckles that were formerly used.

After the rods were tightened, the wall was covered with a 2 in. layer of gunite reinforced with 13 gauge welded wire fabric on 4 in. centers each way.

The roof is of standard flat slab construction supported on 14 in. square columns spaced at 20 ft. centers in both directions.

Cement research in pozzolana cements in the U.S.S.R.

ANONYMOUS. *Cement & Lime Manufacture*, Vol. XIII, No. 12, Dec. 1939, pp. 241-248.
Reviewed by J. C. PEARSON

Some years ago the government set up a committee to study the effect of sea water, sulphate and other aggressive solutions on concretes, with particular reference to the use of pozzolana as a preventive. As a result there is now a considerable production of pozzolanic cement in the U.S.S.R. The article gives brief abstracts of ten papers published by members of the committee, all of them showing the greater resistance of this type of cement to aggressive waters and to most natural waters. Numerous types of pozzolanic materials were used in the tests, including ground quartz, trass, volcanic ash, diatomite, silica wastes (Si-stoff), and basic and acidic blast furnace slags. In these tests considerable attention was given to comparing the accelerated effects of strong sulphate solutions with those of quite weak solutions, and the results were not always parallel. For example, in weak solutions nearly all the common water soluble sulphates had much the same effect, the portland cements gradually deteriorating, whereas they seemed actually to improve the hardening of the pozzolanic cements. In 10% solutions of Na_2SO_4 and $MgSO_4$, however, opposite effects were observed, the former having a more damaging effect on the portland cements, while the latter caused more rapid deterioration of the blended cements.

In studies of comparative resistance to fresh water the most reliable method was found to consist in percolation tests on mortars. Density as well as solubility of the cements is evidently an important factor. Both these factors were stated to be definitely in favor of the pozzolana cements, in which the water resistance was stated to be directly proportional to the reactivity of the hydraulic admixture used in the blend.

In regard to shrinkage, portland cements showed lower shrinkage under air curing at the age of 1 year. In moist or combined curing, however, all pozzolana and slag cements showed lower shrinkage than portlands. (The type of specimens and mixtures is not indicated.) At high temperatures, the pozzolanic cements showed the greater contraction, but at the age of three months pozzolanic cements had greater resistance to temperatures from 600 to 900° C. than portlands, as shown by compressive tests. (Again, the type of specimens and mixtures is not given.)

For mass concrete, the pozzolanic cements have definitely lower heats of hydration and lower solubility, with sufficient strength and impermeability. A special cement for hydraulic structures is now produced by grinding 60 to 70 per cent of clinker with 20 to 25 per cent dry diatomite and 10 to 15 per cent of quartz sand.

Some data are given on the production of rapid hardening slag or pozzolana cements. The early strength of these cements is naturally lower than portlands, and studies were made to find the most suitable clinker to overcome this disadvantage. Experimental cements were burned in the laboratory, and a wide variety of admixtures were included. It was found that good burning was essential for

early strengths, and the necessary chemical composition was as follows: (1) A lime saturation factor varying between 1.00 and 1.05, (2) A silica ratio from 4.00 to 5.00, and (3) An alumina/iron oxide ratio not less than 2.45.

The final paper deals with the values of basic and acidic slags as blending material. It has been generally believed that slags being a ratio of $(CaO + MgO)/(SiO_2 + Al_2O_3)$ less than unity cannot be used for the production of cement. Three slags were studied in which the above ratios were .70, .82 and .58. Ground with slaked lime these slags were shown to possess high reactivity. The rate of strength development under water curing was much higher than under air curing, and in the latter case pronounced reduction in strength occurred in 6 months to 1 year, in the manner characteristic of lime-pozzolana cements. Blended with portland cements, these acidic slags behaved as typical pozzolana cements, except that they showed higher strengths with air curing than with water curing.

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PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

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From JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, Vol. 11, Sept. 1939, to June 1940 and Supplement, Sept. 1940

This is an Index of:

Original contributions to the JOURNAL OF THE AMERICAN CONCRETE INSTITUTE—papers, reports, discussions by subject, title and author.

For the convenience of the reader who is referring to JOURNAL issues rather than to Bound Volumes (in which discussion is assembled following the paper discussed) there is a reference to JOURNAL issue in which discussion appeared. This reference is by supplementary page numbers. If the last page of a paper is 24, the first page of the discussion, published later, is 24-1.

In general, important subjects are classified and indexed under approximately 30 main headings each one appearing in its proper alphabetical order in bold face capital letters—as for instance, ARCHITECTURAL DESIGN. "Tower," and other subjects classified under this head, are indented.

Key words to important subjects, appear, in alphabetical order in addition to the general classification—as for instance "Admixtures" and "Aggregates" each referring to MATERIALS AND TESTS under which all allied references appear, indented. Authors' names and original titles of papers appear in bold face type in proper alphabetical sequence with the subjects, with references to their contributions.

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